Forty-eighth Annual Convention, Portland, Me.
September 17-20, 1929.

Volume 43. JUNE, 1929. $4.00 a Year.
Number 2. Pages 113-237. $1.25 a Number.

M. M. GIBBONS

JOURNAL

OF THE

New England Water Works Association.

ISSUED QUARTERLY.

PUBLISHED BY
THE NEW ENGLAND WATER WORKS ASSOCIATION,
714 Tremont Temple, Boston, Mass.

Entered as second-class matter September 23, 1903, at the Post Office at Boston, Mass., under Act of Congress of March 3, 1879.

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A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

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The initiation fees and annual dues are as follows:

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New England Water Works Association

ORGANIZED 1882.

Vol. XLIII. June, 1929. No. 2.

This Association, as a body, is not responsible for the statements or opinions of any of its members.

THE NEW STORAGE FACILITIES OF THE PAWTUCKET WATER WORKS.

BY FREDERICK C. WILLIAMS.*

[*Read January 9, 1929.*]

The city of Pawtucket, R. I., has recently spent approximately one million dollars on new storage facilities for the Pawtucket Water Works. These water works supply the communities of Pawtucket, Central Falls, Valley Falls, Lonsdale, Berkeley, Ashton, Marieville and Saylesville and the Watchemoket Fire District in the town of East Providence. The population now served is about 125 000. During the past year 3.93 billion gal. of water were supplied. This is at the rate of 10.75 m.g.d. or 86 g.p.d. for each consumer. The population is estimated to increase to about 126 000 by 1930 and 222 000 by 1975 as shown in Table 1.

TABLE 1. — FUTURE WATER DEMAND OF PAWTUCKET WATER WORKS. (1922–1974).

Year.	Population.	Water Consumption.			Number of Services.	Persons per Service.
		m.g.d.	g.p.d. per cap.	g.p.d. per service.		
1922	109 000	8.9	81	733	12 149	9.0
1924	113 000	9.6	85	756	12 700	8.9
1930	126 000	10.8	86	756	14 300	8.8
1940	148 000	13	88	756	17 200	8.6
1950	170 000	15	88	743	20 200	8.4
1960	191 000	17	89	730	23 300	8.2
1970	212 000	19	90	717	26 500	8.0
1975	222 000	20	90	702	28 500	7.8

Source of Supply. The supply is secured by diversion from the Abbott Run, a stream rising in the town of Wrentham, Mass., near Uncas Pond. The stream flows southerly for about 12 miles until it reaches the Blackstone River. Near Diamond Hill, R. I., the stream joins with the east branch of Sneech Brook. The Abbott Run has a drainage area of 27.8 sq. miles above the diversion dam at Happy Hollow. The only controlled storage within the catchment area was at its upper end in the Diamond Hill

*City Engineer, Pawtucket, R. I.

Reservoir created by an earth dam built by the city in 1887. This reservoir has a capacity of 1.456 billion gal.; a water surface of 260 acres; an average depth of 17 ft. and a drainage area above the dam of 8.3 sq. miles. It will thus be seen that only 30 per cent. of the entire drainage area was subject to controlled storage.

Above the intake upon Abbott Run there are the following ponds:

1. *Happy Hollow Pond*, about one and one-quarter miles from the center of Pawtucket, is created by a masonry dam about 10 ft. high, with crest of flashboards at elevation 48.1. Area of pond 23 acres; average depth 4.8 ft., and capacity 36 million gal. Ice is cut upon this pond and stored in a house a short distance above the intake.

2. *Cumberland Mills Pond*, created by a wooden dam rebuilt in 1905, with crest at elevation 62.33, forming a pond with an area of 39 acres; average depth 5.7 ft.; impounding 73 million gal. of water. Ice is cut from this pond, too, and stored in a house a short distance above the dam.

The yield of Abbott Run has been measured at this point carefully and continuously since the year 1907 by means of weirs and devices for measuring and recording the flow.

3. *Adamsdale Pond*, created by a masonry dam giving an area of 26 acres and capacity of 24 million gal. A small yarn mill utilizes the power at this dam.

4. *Howards Pond*, area 12 acres, capacity 8 million gal. Created by masonry dam with wooden flashboards. A small gristmill develops power at this dam.

5. *Rawson's Pond*, created by masonry dam with wooden flashboards. Area 31 acres, capacity 42 million gal. There is a large ice plant upon this pond, at the dam.

6. *Arnolds Mills Pond*, area 2 acres; capacity probably about 6 million gal. A small gristmill develops power at this dam.

Yield of the Abbott Run. Beginning with the year 1908 the late George A. Carpenter, City Engineer, maintained a continuous record of the yield of the Abbott Run at the Cumberland Mills Dam. The measuring station was equipped with two sharp-crested weirs, one for large flows and one for small flows. The average daily yield during the record period has been approximately 25.5 m.g.d.

During the year 1910–1911 the cumulative run-off from land area only was very low, and the following figures show the storage required to meet various drafts.

Draft, m.g.d.	Storage Required, m.g.
8	800
10	1 190
12	1 500
14	1 900
16	2 340
18	3 000
20	3 650
22	4 400
24	5 050
25.5	5 500

Table 2 gives the average yield of the drainage area since the inception of the record in 1908.

TABLE 2. — AVERAGE MONTHLY YIELD OF LAND SURFACE 1908–1928 INCLUSIVE. ABBOTT RUN WATERSHED.

Month.	Average for Month, m.g.	c.f.s. per sq. mile.	Month.	Average for Month, m.g.	c.f.s. per sq. mile.
Jan.	1 034.122	1.970	Jul.	425.504	0.811
Feb.	1 091.545	2.280	Aug.	276.980	0.528
Mar.	1 834.312	3.496	Sep.	316.388	0.623
Apr.	1 449.094	2.853	Oct.	294.689	0.562
May	1 045.732	1.993	Nov.	443.532	0.873
Jun.	570.175	1.123	Dec.	734.794	1.400

In the summer of 1923 the capacity of the existing storage works was severely tested. The average consumption mounted to 10.1 m.g.d., and during the summer and fall months a demand of over 13 m.g.d. occurred on many days. The possibility of a year of low rainfall during the summer and fall months similar to the year 1923 being followed by a second year of low rainfall caused Isaac Gill, the Commissioner of Public Works, to study every possible means of reducing consumption and increasing storage. The per capita consumption, however, was low; about 94 per cent. of all services were metered, and it seemed impracticable to reduce the consumption by any considerable amount.

The consulting services of Metcalf and Eddy, Boston, were secured to make plans for the development of additional storage facilities, and at the same time steps were taken to secure immediate relief by cutting off one of the outside sections to which the city was supplying water. Notice was served upon that section of the town of East Providence, incorporated as the Watchemoket Fire District, that on and after May 21, 1924, the city would be unable to furnish them with water. The actual cutting-off of the supply, however, was stopped by a court order.

Recommended Development. The report of Metcalf and Eddy recommended an increase of the present supply by progressively building additional dams upon the Abbott Run by means of which the flood waters could be stored and subsequently drawn upon in such a way as to equalize the flow and make available at least three-fourths of the average daily capacity of the stream which is about 25.5 m.g.d. The report outlined the necessity of the construction of six reservoirs during the 50-year period from 1925 to 1975 under consideration and proposed storage reservoirs at the following sites:

1. *Arnolds Mills.* Tributary drainage area 18.9 sq. miles.

a. Dam 31.5 ft. high; spillway at elevation 155; storage capacity 680 m.g.; water surface 185 acres; average depth 11.0 ft.; total cost $358 000.

b. Dam 36 ft. high; spillway at elevation 160; storage capacity 1 020 m.g.; water surface 240 acres; average depth 13.5 ft.; total cost $481 000.

2. *Diamond Hill Dam.* Drainage area 8.3 sq. miles.

a. Crest raised 10 ft.; spillway raised 10.3 ft. to elevation 186.5; storage capacity 2 440 m.g.; water surface 340 acres; average depth 21.8 ft.; total cost $312 000.

b. Crest raised 20 ft.; spillway crest at elevation 197; storage capacity 3 720 m.g.; water surface 410 acres; average depth 27.6 ft.; total cost $614 000.

3. *Sneech Brook*, west of road crossing Diamond Hill Plain; drainage area 4.7 sq. miles.

a. Dam 25.5 ft. high; spillway at elevation 180; storage capacity 560 m.g.; water surface 170 acres; average depth 10 ft.; total cost $459 000.

b. Dam 30.5 ft. high; spillway at elevation 185; storage capacity 880 m.g.; water surface 230 acres; average depth 12 ft.; total cost $600 000.

c. Dam 40 ft. high; spillway at elevation 195; storage capacity 1 740 m.g.; water surface 300 acres; average depth 17.5 ft.; total cost $904 000.

4. *Rawson's Dam*, just below the site of the present dam. Dam 21 ft. high; spillway at elevation 120; storage 190 m.g.; water surface 72 acres; average depth 8.1 ft.; total cost $219 000.

5. *Site between Adamsdale and Howards Mill Ponds.* This site was not studied in detail, because of its location in Massachusetts outside of the state of Rhode Island, and the impossibility of getting legislative authority to condemn land, etc., in Massachusetts.

6. *Happy Hollow Site*, at a point about 2 000 ft. upstream from the present Happy Hollow Dam; dam 22 ft. high; spillway at elevation 66; storage, including Cumberland Mills Pond, 314 m.g.; water surface 140 acres; average depth 7 ft.; total cost $367 000.

The last stage of the first step in the progressive development was to be completed by 1934 and the second step, that of raising the Diamond Hill Dam 10 ft. to give 984 million gal. additional capacity was to be completed by 1939. The building of the Happy Hollow Reservoir was scheduled for 1928. This reservoir was regarded as an equalizing pool to take care of the daily fluctuations in draft and to eliminate the suction lift upon the pumps.

The Arnolds Mills site was selected for the first reservoir because of the advantage to be derived from the development of storage upon a new section of the main drainage area and also because the land to be flowed was all in Rhode Island and no interstate difficulties would arise. The dam site was one-half mile downstream from the old dam at Diamond Hill.

Topography of the Drainage Basin. The topography of the basin is hilly in character with low hills and gently rolling glacial plains. The range of elevations upon the basin is from elevation 48 to about 400. The slopes are flat. The river valley has no distinctive fluvial character. In the basin there are to be found many swampy areas, and in many instances these swamps border small glacial lakes. The low hills are made up largely of shales, conglomerate sandstones and red beds which have been overlain by glacial till.

A large proportion of the soil is of the stony loam type. It is a brown or grey, loose sandy loam, containing widely varying amounts of gravel and

rounded boulders up to one foot in diameter. The subsoil to an indefinite depth consists of glacial till of coarse, medium and fine grades, containing as much as 60 per cent. of rounded roughly-stratified boulders and large gravel, forming a very thoroughly drained and unretentive stratum. At various points in the basin unstratified deposits of till were found with all the silt and rock flour washed away.

The city acquired during 1925 sixteen parcels of land totaling approximately 475 acres for the site of the new reservoir, and in the fall work was started on a very thorough investigation of the foundation conditions.

Testing Foundations. During the summer of 1925 a very careful investigation was made by means of test pits, and wash and core borings of the site of the new dam. Some 42 wash borings were made to rock and 4 core borings were made to a depth of 40 ft. below the surface and then 12 ft. into rock.

In general the whole floor of the valley was found to be covered with gravel, sand and rock flour, some of which was quite pervious. The bed rock was found to be a red conglomerate similar to the green or grey pudding stones of southern Massachusetts and Rhode Island. The rock varied greatly in quality and character. At some points it was extremely friable and contained many seams and joint planes; at other points it became a very dense sandstone.

The elevation of the bed rock varied greatly. At the easterly abutment of the dam, the rock came within 6 ft. of the surface, then it dipped very abruptly to the eastward and 300 ft. beyond the east abutment no rock was found at a depth of 50 ft. From the east abutment westerly the bed rock dipped quite sharply to the bed of the Abbott Run where the core borings indicated that it was some 15 ft. below the bed of the stream. Rock then rose on the other bank to a point within 6 ft. of the ground surface. It then dipped gently to the westward and continued at varying elevations until, at a point 1 300 ft. from the east abutment, it was again within 6 ft. of the ground surface. It then dipped very abruptly to the westward and after a distance of 200 ft. the core drill failed to locate the bed rock. This condition continued to the westerly abutment at station 41+80 or 2 980 ft. from the easterly abutment.

The wash borings and test pits indicated that whereas some pervious material might be encountered, the foundation strata would be reasonably tight, as they consisted either of bed rock or extremely fine sand.

In addition to the investigation of the foundation strata, the entire valley area was searched for material suitable for the embankment structure of the dam. The results of this search were very discouraging. No clay was found, and the loam was of a very light sandy nature and practically incapable of consolidation. The only materials encountered were fine and coarse sand consisting almost entirely of quartz grains, and it appeared that it would be necessary to transport the embankment material from some other location. A further search, however, uncovered a small

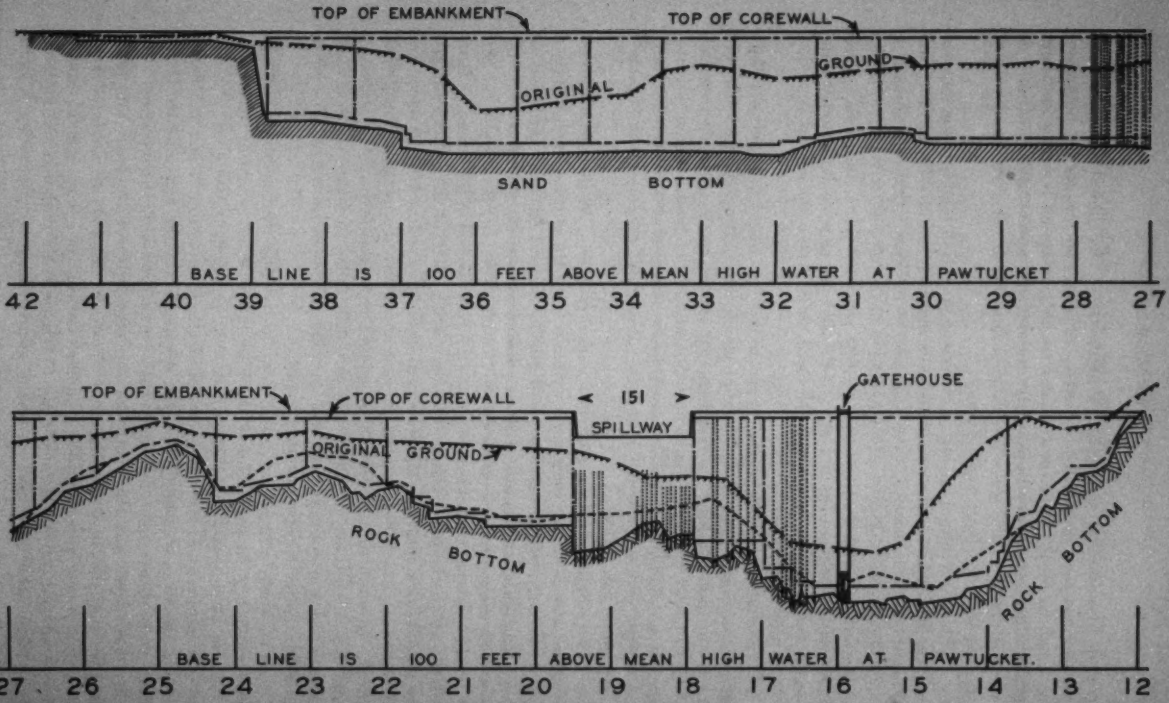
TOP OF EMBANKMENT
TOP OF COREWALL
GROUND
ORIGINAL
SAND
BOTTOM
BASE LINE IS 100 FEET ABOVE MEAN HIGH WATER AT PAWTUCKET
42 41 40 39 38 37 36 35 34 33 32 31 30 29 28 27
TOP OF EMBANKMENT
TOP OF COREWALL
151
GATEHOUSE
SPILLWAY
ORIGINAL GROUND
ROCK BOTTOM
ROCK BOTTOM
BASE LINE IS 100 FEET ABOVE MEAN HIGH WATER AT PAWTUCKET.
27 26 25 24 23 22 21 20 19 18 17 16 15 14 13 12

FIG. 1. — ELEVATION OF ARNOLDS MILLS DAM.

deposit of rock flour in a lateral valley leading to the site of the east dike. From the volume of the deposit it appeared that by using all the loam available and carefully mixing the rock flour with sand and gravel enough material might be secured to complete the embankment.

The Arnolds Mills Dam.

Design. The detailed studies under the first step in the recommended development covered the following: (*a*) An earth-fill dam and dike 2 900 ft. long with a maximum height of 36 ft.; (*b*) a small dam 530 ft. long with a maximum height of 19 ft.; (*c*) the reconstruction of the N. Y., N. H. & H. Railroad line across the reservoir basin; and (*d*) the protection of the Diamond Hill Dam from the wash of the new reservoir.

The studies made indicated that with the crest of the spillway at elevation 155, the maximum safe yield of the Arnolds Mills Reservoir would be 4.5 m.g.d. and with the crest at elevation 160, the maximum safe yield would be 5.6 m.g.d. These figures were based on a tributary drainage area for the reservoir of 10.6 sq. miles.

It was felt that the difference between the estimated cost of $358 000 for a dam with spillway at elevation 155 and $461 000 for a dam with spillway at elevation 160 was comparatively small when considered on the basis of an increase of 340 million gal. in capacity, and it was decided that the project should be undertaken for the higher dam. Contracts were therefore prepared for the higher structure.

Due to the peculiar shape of the river valley at the reservoir site it was necessary to provide an extremely long dam. The dam is a rolled embankment structure approximately 3 000 ft. long on top with a maximum height above the bed of the stream of approximately 36 ft. A central core of reinforced concrete extends the entire length of the dam. This core is founded on bed rock for approximately one-half of the length of the dam. As already stated, in the western half of the dam the core wall is founded on very fine sand. At points where coarser material was found wooden cut-off sheathing was provided for. (Fig. 1.)

The concrete core wall has a maximum vertical height of approximately 50 ft. At this height it has a footing 7 ft. wide; the base is 3.5 ft. in width and reduces in thickness to 12 in. at the top.

The dam has a top width of 18 ft. and a slope of 1 on 2 on both the upstream and downstream sides. A 6-ft. berm has been provided at elevation 146 on the downstream slope. The upstream toe consists of a heavy rock fill for a vertical height of approximately 14 ft. to prevent any movement of the embankment. On the downstream toe a similar protection is provided to a vertical height of 5 ft. to prevent erosion of the embankment from wave action from the mill pond immediately below.

The spillway is a gravity section 151 ft. long placed to the westward of the main dam on the rock shoulder that approached to within 6 to 12 ft.

of the surface. The spillway section is built entirely in cut and has a base width of 28 ft. and a maximum height of 30 ft.

A stilling pool 20 ft. wide and of the same length as the spillway section has been provided at the base of the dam. The stilling pool is formed by a buffer dam 14 ft. wide and 5 ft. high. An 18-in. vitrified drain pipe has been provided through the buffer dam for draining the stilling pool at times when the spillway is not in action.

Immediately below the spillway section, a channel was excavated to the main stream. A concrete retaining wall has been provided on the westerly side of this channel for approximately 364 ft. On the easterly side of the channel the retaining wall is approximately 200 ft. long.

In addition to the main valley, a relatively shallow lateral valley running to the eastward of the reservoir basin had to be cut off by means of a dike similar to the main dam in construction. This structure is approximately 530 ft. long, 138 ft. wide at the base and 19 ft. high. The additional width of base was due to the fact that it was necessary to carry a road leading to North Attleboro on the toe of the dike.

The spillway section has been designed to have a discharge capacity of 4 032 c.f.s. with a head of 4 ft. This is equivalent to 214 c.f.s. per sq. mile, or twice the maximum flood flow of the stream as evidenced by a record of 50 years. A storm of sufficient intensity to cause the spillway to overflow with a head of 4 ft. would occur probably once in a hundred-year period. At considerably rarer intervals a head of 5 ft. might be reached giving a discharge of 5 790 c.f.s. or 306 c.f.s. per sq. mile.

The record of the higher rates of flow at Diamond Hill for a period of nearly 50 years is as follows:

Flood of.	c.f.s. per sq. mile.
November 27, 1885	27*
February 12, 1886	80
March 11, 1901	30*
March 1, 1902	108
March 8, 1904	49
January 7, 1905	15*
March 16, 1907	10*
January 21, 1910	12*

*Average rate for 24 hours.

The rates of flow, except as otherwise indicated, are the maximum rates during the progress of the flood.

Preliminary Construction. The first step in actual construction of the new project was the extension of the toe of the Diamond Hill Dam. A contract was entered into with John J. McHale & Sons of Pawtucket, R. I., for this work. The principal items in this contract consisted of 35 000

cu. yd. of excavated material placed at the toe of the dam, 5 200 cu. yd. of heavy rock revetment and a new outlet structure. The contract cost of this work totaled $64 932.57. The work was started on October 1, 1925, and was practically completed at the end of January, 1926. The material used consisted of a combination of gravel, sand and loam. Practically all the material was placed by means of bottom-dump wagons, some 18 being employed at this point. A five-ton tractor with a manually-operated bull-dozer or strike board was used to level the material to the required six inches in thickness. Fill was compacted by a sprinkler and a three-ton corrugated roller drawn by the tractor. In removing the muck and loam from the site of the work it was discovered that a blind stone drain had been placed near the west abutment of the dam. This drain had to be extended and the water led away through the new revetment.

The heavy rock revetment consisted of waste granite from the old Diamond Hill quarry. This material was ideally suited for the protection of the rolled embankment and was graded from blocks 3 to 4 ft. on side down to small stones, about 3 or 4 in. in diameter. Considerable work was done in extending the existing spillway channel and in securing a uniform benching of the rock so that an equal distribution of flow over the spillway channel could be obtained during time of flood.

During the progress of the work on the Diamond Hill Dam it was decided in view of the low yield for the fall months of 1925 and the danger of the beginning of a dry cycle that flashboards should be provided for the spillways of this structure. The flashboards were 18 in. high and gave an additional storage capacity of 132 million gal. These flashboards were designed in such a manner that in case it was necessary to lower the reservoir quickly the pins supporting the boards could be pulled. In order to be sure that the device would work properly the pins were pulled with 12 in. of water coming over the flashboards. As it worked out the addition of these boards probably saved the system from a very serious water shortage, because during the first seven months of 1927 southern New England suffered from the dryest period since 1839, the total rainfall being only about 16 in., practically 10 in. below normal.

As the supply in the Diamond Hill Reservoir was drawn upon, it was noted that in addition to the 132 million gal. actually above the spillway crest, something like 100 million gal. were evidently stored in the porous strata of the adjacent hills. This meant that during 1927 the water was never below the spillway crest of the Diamond Hill Dam. In the average year the water drops below the crest by the first of July.

All the work undertaken at the Diamond Hill Dam, namely, the construction of the rough embankment with coarse rock revetment on the downstream slope, the extension and deepening of the existing spillway channel, the extension of the outlet piping and the construction of the concrete pitometer chamber for the piping, was chargeable to the future raising of the Diamond Hill Dam, although it must be understood that the

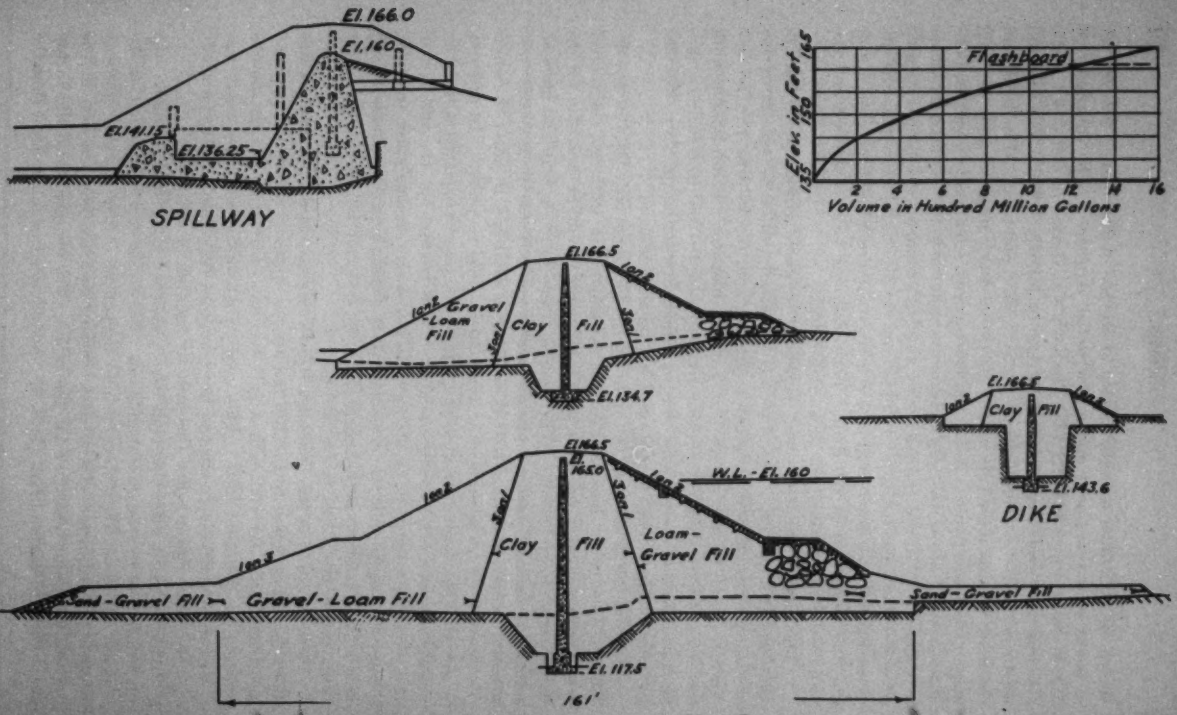

FIG. 2. — CROSS SECTIONS OF ARNOLDS MILLS DAM.

heavy rock revetment was to prevent erosion by wave action and the slipping of the embankment due to its saturation upon the downstream side by the waters of the Arnolds Mills Reservoir.

Although the new work has been practically flooded due to high water in the Arnolds Mills Reservoir, levels indicate that there has been no settlement of the rolled embankment excepting near the pitometer chamber where the usual method of distributing and rolling the material could not be followed.

Main Construction Methods. Four bids were received for the main construction work on March 17, 1926. The prices ranged from approximately $415 500 to $575 000. The contract was awarded to John J. McHale & Sons of Pawtucket, R. I., on the same date.

Actual construction work was started early in April, 1926. The drawings and specifications called for a core wall trench 6 ft. in width, and after the dam site had been stripped of loam and vegetable matter, work was started on this trench. The trench excavation quickly revealed that the subsoil to an indefinite depth consisted of uniform white sand of coarse, medium and fine grains together with a stratum of coarse sand containing as much as 60 per cent. of rounded boulders and large gravel forming a very thoroughly drained and unretentive deposit. The controlling dimensions of the structure are shown in Figures 1 and 2. As the work progressed this stratum was found to extend in all directions at varying elevations. At some spots it almost "pinched out", its width varying from 18 in. to 6 ft. With this development it was felt that a 6-ft. width of core trench was altogether too favorable for seepage conditions and the design was changed to provide for a core trench 15 ft. in width. This change in design and method of construction meant a large increase in the amount of excavation and also in the amount of material to be backfilled.

The major items of the contractor's plant consisted of 10 ten-ton trucks; 3 three-quarter-yard steam shovels; 2 ten-ton tractors; 1 clam-shell derrick; 2 embankment rollers; 1 complete concrete aggregate batching plant; 1 three-quarter-yard drag line for handling aggregate; 1 crushing plant; 1 portable air compressor and grouting outfit; 2 three-yard trailers; 1 one-half-yard concrete mixer of the self-propelling paver type and 1 incline belt conveyor.

One of the interesting features of construction was the relation of mechanical equipment to the number of men employed. The value of the construction equipment used was about $150 000 and the average number of workmen on the payroll was about 40. This represented $3 700 of equipment for each man employed; an exceptionally high ratio even on a job where the time element was of the highest importance.

The contractor constructed a cofferdam large enough to include nearly all of the dam in the main river valley by the simple expedient of digging a diversion channel about 500 ft. long and 20 ft. wide and then constructing two coffer arms across the whole stream above and below the work. (See

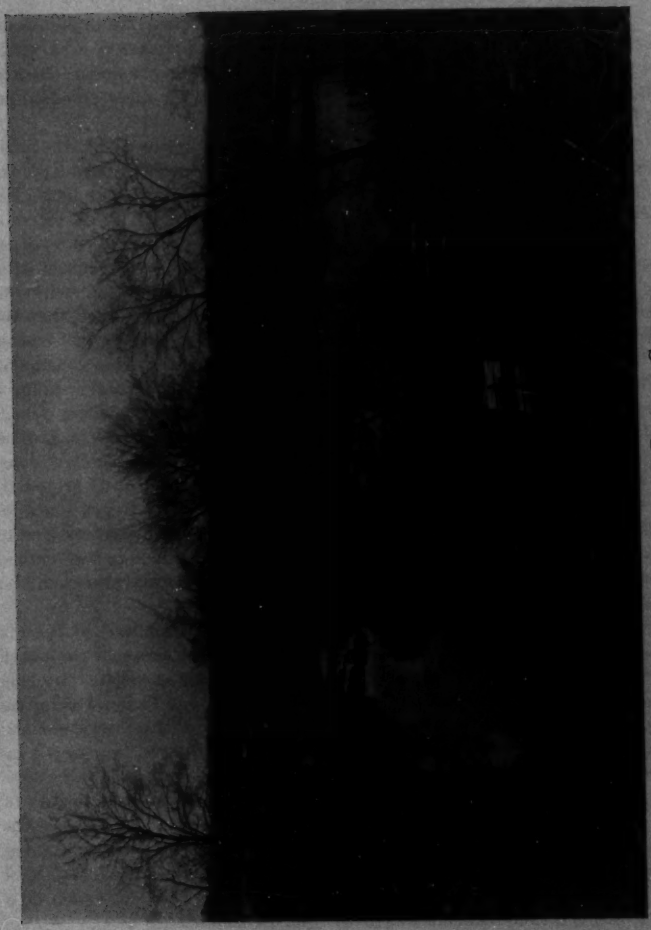

FIG. 3. COFFERDAM AND DIVERSION CHANNEL.

Fig. 3.) The water within the cofferdam area ran out almost entirely, and very little pumping was necessary. A large amount of silt, black mud and decaying vegetable matter was removed from the area within the cofferdam. The diversion channel was used until the blow-off pipes were permanently constructed through the gatehouse foundation.

Steam shovels were used for stripping the site and for excavating the core trench wherever possible. When the depth of the core trench became too great, it was necessary to sheath the side walls and handle the excavated material by means of buckets. Removing the disintegrated rock immediately above the bed rock surface was accomplished by means of picks and shovels. The key way, 12 to 18 in. in depth, was then cut into the foundation rock by means of compressed air drills and by barring and wedging.

Core Wall. A thin concrete core wall varying in thickness from 3.5 ft. to 1 ft. was provided throughout the length of the embankment structures. The reinforcing consisted of ⅝-in. round bars 18 in. on centers in both directions and in both faces of the wall.

The core wall was placed in 72-ft. sections using unit panels 12 ft. long and 8 ft. high. Relatively long sections of the core trench were left open in order to secure as long a section of embankment as possible. Due to the location of the spillway and the outcropping of the rock near the middle point of the dam, it was possible to construct the main dam as one unit and the dike from the spillway westerly to the outcrop at the middle point, as another unit. (See Fig. 4.)

The concrete for the core wall was mixed in a half-yard paving mixer on the bank and the concrete was deposited in the form by means of chutes. Frequent moving was necessary in order that the concrete might be deposited to a uniform depth. As the work progressed and the rolled fill reached above the original surface of the ground, it was necessary to provide an incline conveyor for placing the concrete. The combination of the portable mixer and the portable conveyor worked extremely well. Unusual pains were taken in order that the concrete in the core wall might be as dense as obtainable and that honeycombed spots might be eliminated.

Rolled Embankment. The embankment materials were placed in layers from 6 to 12 in. in thickness by ten-ton trucks and trailers while dumping, and kept level by bulldozers, hydraulically operated, attached to 2 ten-ton Caterpillar tractors. The contractor at first attempted to roll the embankment by means of an eight-ton gasoline roller and a loose ring, corrugated roller. From careful observation it was determined that the ten-ton tractors and the constant rolling of the heavily loaded trucks did a far better job than the rollers. The rollers gave a very fine-appearing surface, but when test pits were dug in the areas entirely compacted by the rollers and in the areas compacted by trucks and Caterpillars alone, it was found that the vibrating effect of the Caterpillars had secured a remarkable degree of consolidation and settlement.

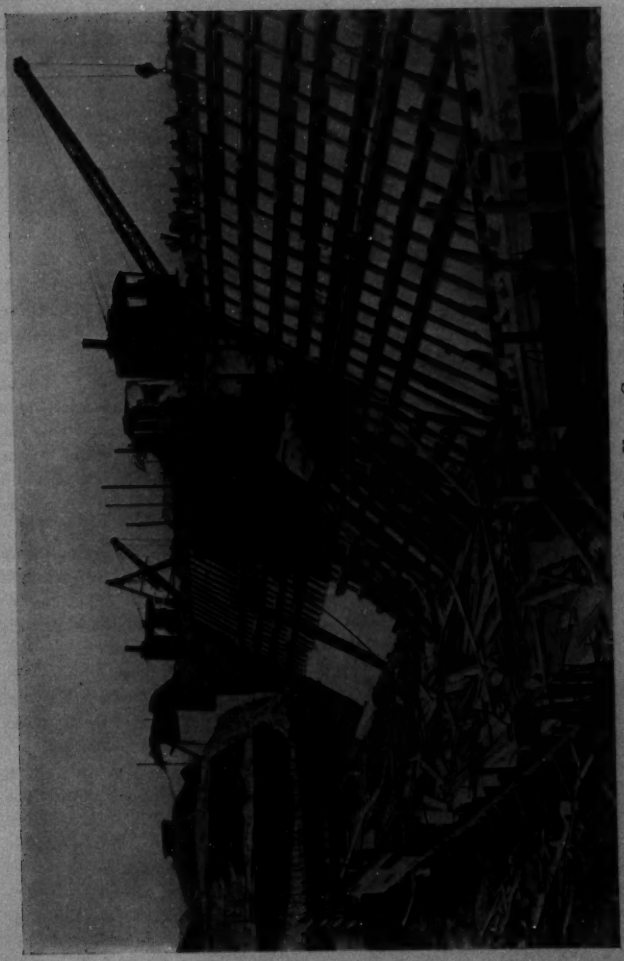

FIG. 4. — SPILLWAY SECTION UNDER CONSTRUCTION.

As the work of placing the rolled embankment progressed it was seen that the supply of rock flour would not be sufficient, and it became imperative to secure additional fine material. As a last resort it was decided to use some of the loam back of the spillway section of the dam. The operation of removing the loam had hardly been commenced before it was discovered that the underlying strata in this particular section consisted wholly of extremely fine rock flour. From then on this new borrow pit was used entirely for securing impervious material even though it was within 500 ft. of the axis of the dam, this limit not being exceeded, however, because of the apparent perviousness of the material.

In placing the concrete in the spillway section, construction joints were provided every 35 ft., and in addition to the usual key ways, sheet metal cut-off plates were introduced near the upstream side of the structure in every joint. In order that there should be no discoloration of the concrete surface the plate for the upper three feet consisted of a ½-in. layer of Elastite. During the winter these joints showed considerable movement, but they remained tight.

The concrete was placed in the spillway forms by means of buckets or chutes in courses approximately 6 ft. thick.

Testing and Grouting the Foundation. In the easterly half of the dam which was founded on rock every protection was taken to make the foundation as tight as possible. Test holes were drilled into the rock and tested under both air and water pressure. If a hole showed any appreciable leakage it was immediately grouted. In the spillway section, due to the friable and seamy nature of the rock, it was necessary to load the foundation with one or two courses of concrete before the testing and grouting could be undertaken.

Gatehouse. The original boring indicated that good rock foundation would be found at elevation 106 for the gatehouse. As the excavation progressed it was found that good rock bottom could be obtained at elevation 116.5. A heavy foundation slab approximately 2 ft. in thickness was placed in order to carry the gatehouse walls. The circular foundation walls of the gatehouse provided for a well 13 ft. in diameter, and the walls were 12 in. thick.

The lower intake pipe through the dam consisted of 48-in. precast lock-joint concrete pipe and the outlet pipe consisted of a 60-in. pipe of the same design. Both of these pipes were carried on concrete cradles. Each length of pipe weighed over seven tons, and was unloaded from flat cars by means of a crane and transported to the site of the work in ten-ton dump trucks. Here it was unloaded by simply lifting the truck body and allowing the pipe to slide out onto soft ground and a timber cradle. The sections were then placed in the trench by means of a steam shovel. During the placing of the pipe one section was dropped due to a defective hook on a wire-rope sling, but the pipe was not damaged.

The center line of these pipes was placed at elevation 130. A high-

FIG. 5. — SPILLWAY AND EMBANKMENT.

level intake was provided at elevation 147.35. This inlet consists of 24-in. cast-iron pipe carried on a concrete cradle. In addition to the 60-in. outlet pipe through the dam a 36-in. cast-iron line was provided for the normal flows to the city. This line contained a Venturi meter, and a reinforced-concrete chamber with a brick access-manhole was constructed around the meter throat. All pipes passing through the rough embankment were provided with concrete collars. Three 36-in. valves and one 24-in. valve were provided in the gatehouse substructure for control of the blow-off, meter and intake pipes.

The gatehouse superstructure was constructed of brick with artificial limestone trim and a concrete roof. The superstructure contains the

FIG. 6. — SPILLWAY AND POOL.

operating stands for the valves, the Venturi meter recorder and the water stage register.

Embankment Paving. The upstream slope of the main dam and dikes was paved with 12-in. stone paving, concrete grouted to a point within 7 ft. of the top of the embankment. The area above was provided with 8-in. dry stone-paving. Concrete paving-stops were constructed at the foot of the paving and again at a point near the normal water surface. Weep slots 18 in. wide at 15-ft. intervals were provided throughout the paved area. Before the paving was placed the slopes were thoroughly consolidated with compressed-air tampers.

Results. The outlet gates were closed for the final filling of the reservoir on December 22, 1927. Water went over the spillway for the first time on February 17, 1928, the reservoir then containing 1 165 600 000 gal.,

having reached an elevation of 160 ft. above sea level. Figure 5 shows the completed spillway section with flashboards in place.

No noticeable settlement of the embankment has been observed throughout its entire length. With 15 in. of water on the spillway the section of the dam founded on rock appeared to be remarkably tight. The westerly section founded on fine sand has shown the expected percolation which has not been large in volume and has been taken care of by a 12-in. intercepting drain running roughly parallel to the axis of the dam and dike. Careful observation has indicated that a considerable part of this percolation comes through the high land back of the westerly abutment. The same condition of underflow exists in the saddle 200 ft. east of the east abutment. The flow appears, however, at such a distance downstream from the embankment of the main dam as to be inconsequential. The total seepage measured at a point 1 000 ft. below the dam by means of a V Notch Weir, amounts to about 190 000 gal. in 24 hours. Figure 6 shows the main dam east of the spillway.

Raising Railroad Embankment. The raising of the railroad embankment and providing a new plate-girder bridge with concrete abutments was undertaken by the N. Y., N. H. & H. Railroad under an agreement with the city of Pawtucket wherein the city of Pawtucket guaranteed the payment of the work. The embankment was raised some 10 ft., and the slopes were protected by very heavy stone riprap. The cost of this work was $73 415.77.

Reforestation. In the spring of 1927 the city established a municipal nursery at Diamond Hill as a source of supply for watershed planting. In that year 100 000 two-year old seedlings of white pine and 3 000 two-year old seedlings of red pine were placed in the nursery. In the spring of 1928 approximately 80 000 of these trees were set out around the new Arnolds Mills Reservoir. Twenty thousand white pines were left in the nursery. One hundred thousand two-year old seedlings of white spruce and 50 000 red pine seedlings were placed in the nursery. In addition 3 000 four-year old Norway spruce trees were set directly in the field.

It is our intention this spring to plant out the 20 000 (now four-year old) white pines and add perhaps 100 000 red and white pine seedlings to the nursery. These trees are now in a most flourishing condition.

Project Costs. The principal items in the first contract consist of the following quantities and contract prices:

Earth excavation, 85 cu. yd. @ $0.60	$51.00
Excavation of loam, 5 304 cu. yd. @ $1.50	7 956.00
Excavation from pit, 29 273 cu. yd. @ $1.075	31 468.47
Rock excavation, 611 cu. yd. @ $10.00	6 110.00
Rock revetment, 5 197 cu. yd. @ $2.04	10 601.88
Excavated earth hauled, 1 740 cu. yd. @ $0.03	52.20
Grassing earth surface, 8 450 sq. yd. @ $0.05	422.50
Class A concrete, 57.445 cu. yd. @ $13.50	775.50
Outlet structure, lump sum	6 000.00

Stone paving, 245 sq. yd. @ $0.70	$171.50
Paving in gutters, 36.8 sq. yd. @ $1.00	36.80
Stream control, lump sum	1 000.00
Cleaning up, lump sum	150.00
Extra work	136.72
Total	$64 932.57

The principal items in the second contract consist of the following quantities and contract prices:

Clearing and burning of trees, logs, leaves, etc., 263 acres @ $49.00	$12 887.00
Grubbing stumps and roots, 18.3 acres @ $191.00	3 495.30
Collecting and burning logs, leaves, etc., lump sum	1 500.00
Earth excavation for pipe and drainage trenches, 5 565 cu. yd. @ $0.90	5 008.50
Earth excavation from core wall trench, 0 to 8 ft., 39 133 cu. yd. @ $1.50	58 699.50
Earth excavation from core wall trench, 8 to 16 ft., 11 396 cu. yd. @ $4.00	45 584.00
Earth excavation from core wall trench, below 16 ft., 4 221 cu yd. @ $7.00	29 547.00
Excavation of loam, subsoil or earth, 14 351 cu. yd. @ $1.25	17 938.75
Earth excavation from borrow pits, 100 365 cu. yd. @ $1.25	125 456.25
Mixing of soil for most impervious material, 3 798 cu. yd. @ $0.90	3 418.20
Rock excavation in trench for core wall, etc., 101 cu. yd. @ $6.00	606.00
Rock excavation within spillway weir site, 3 280 cu. yd. @ $3.75	12 300.00
Handling and removing boulders exceeding ½ cu. yd., 37 cu. yd. @ $4.50	166.50
Excavated material hauled 200 ft. over free limit, 92 081 cu. yd. @ $0.02 (weighted mean)	1 841.62
Grassing earth surfaces, 24 587 sq. yd. @ $0.27	6 638.49
Class A concrete in gatehouse, 171 cu. yd. @ $45.00	7 695.00
Class A concrete in spillway wall and foundation, 1 651 cu. yd. @ $16.00	26 416.00
Class A concrete in inlet and outlet structures, 70 cu. yd. @ $30.00	2 100.00
Class A concrete in core wall of dam and dikes, 6 673 cu. yd. @ $19.50	130 123.50
Class B concrete in spillway weir, 1 282 cu. yd. @ $11.00	14 102.00
Class B concrete in footings for paving, 327 cu. yd. @ $16.00	5 232.00
Class C concrete in pipe cradles, etc., 87 cu. yd. @ $13.50	1 174.50
Hydrated lime in masonry, 9 850 cu. yd. @ $0.50	4 925.00
Steel reinforcing bars for concrete in place, 344 856 lb. @ $0.05	17 242.80
Hauling, erecting and jointing cast-iron pipe, valves, etc., lump sum	2 400.00
Furnishing and laying precast concrete pipe 48-in., 78 lin. ft. @ $24.00	1 872.00
60-in., 72 lin. ft. @ $28.00	2 016.00
8-in. dry stone paving laid in screened gravel, 2 129 sq. yd. @ $1.50	3 193.50
12-in. stone paving in screened gravel with concrete joints, 4 536 sq. yd. @ $3.00	13 608.00
Stone paving in gutters, 59 sq. yd. @ $1.00	59.00
Gatehouse superstructure, lump sum	2 500.00
Venturi meter chamber, lump sum	800.00
4½-in., 3-ply sheet piling in place, 2.242 M ft. B.M. @ $200.00	448.40
Reconstructing North Attleboro Road, 700 lin. ft. @ $3.53	2 471.00
Stream control during construction, lump sum	6 000.00
Cleaning up, lump sum	1 000.00
Lumber left in place, 13 230 M ft. B.M. @ $65.00	859.95
Extra work	9 434.98
Total	$580 760.74

The entire cost of the project is divided as follows:

Acquisition of land, water privileges and flowage damages	$160 992.70
Rock revetment and fill at Diamond Hill Dam	69 507.73
Arnolds Mills Dam and Dike	592 270.16
Raising railroad embankment	73 415.77
Road changes	12 421.00
Office, engineering and transportation	59 088.41
Reforestation	3 700.80
Total	$971 396.57

Personnel. Plans and specifications were prepared for the work by Metcalf and Eddy, Consulting Engineers of Boston, Mass. A separate engineering organization was created for handling the project with Frederick C. Williams in direct charge as Chief Engineer. The project was built under the direction of Isaac Gill, Commissioner of Public Works of Pawtucket, R. I., with Byron C. Bussey, Assistant Commissioner, in general charge.

SOME DISTRIBUTION PROBLEMS.*

BY FRED B. NELSON.†

[*Read September 19, 1928.*]

In providing and maintaining serviceable pressures in a large distribution system the problems are many and the factors involved extremely varied by local conditions. No attempt will therefore be made to cover the field in a comprehensive way in this paper, which will confine itself to a discussion of some of the distribution problems encountered in New York City. The problems to be considered are as old as the art of water supply. They are up for solution in all cities, their importance being varied only by local conditions.

In New York City the introduction of the Catskill supply, delivered by gravity at a much higher head than the original Croton and other supplies, made possible extensive improvements of pressure and modifications and extensions of service zones and involved many radical changes in distribution. These were the local conditions for New York City that added materially to those problems, common to all cities, of providing adequate distribution to meet the present and prospective demands of sectional development, population increase, and other factors.

In New York City the Croton supply is delivered by gravity at a head of 133 ft. above sea level at Jerome Park Reservoir near the north end of the city and is carried southward through original aqueduct lines terminating in Central Park Reservoir at 118 ft. above sea level. The Catskill supply is delivered by gravity at 293 ft. above sea level at the north city line and from there is conveyed by 18 miles of deep rock tunnel through the west side of Bronx Borough and throughout the elongated island of Manhattan to Brooklyn. This Catskill supply is available for these two boroughs from 18 riser shafts along the line of the tunnel, instead of having to be conveyed southward by trunk lines as with the old Croton system.

To correspond with the topography of the boroughs of Manhattan and the Bronx, the system includes four service zones, supplied under heads varying according to the ground elevation included in each. These are: (1) *low service* supplied by Croton water under gravity head, (2) *intermediate service* supplied by pumped Croton water, together with Catskill water under reduced and regulated head, (3) *high service* supplied by Catskill water by gravity or under full head of the Catskill tunnel and (4) *tower service* of Manhattan supplied with Catskill water and Croton water pumped to about 50 ft. above the available Catskill head for adequately serving the area of highest elevation.

*A paper presented at the Superintendents' Conference.
†New York Department of Water Supply, Gas & Electricity.

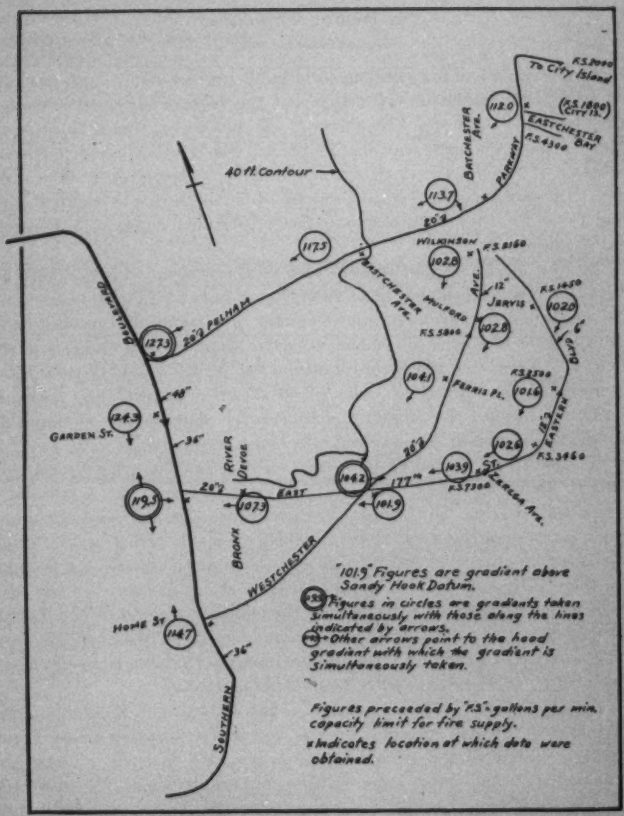

FIG. 1.— HYDRAULIC GRADIENT SURVEY OF EAST BRONX LOW-SERVICE AREA.

In general the aim has been to utilize the original Croton gravity supply as fully as possible, reserving the Catskill supply for the higher areas and to save pumping. Unnecessary extension of Catskill service into areas where use of Croton gravity service was feasible by proper changes or reinforcement in the distribution system has been avoided.

East Bronx Low Service. This feature was a factor in a recent problem of improving the supply to roughly 10 sq. miles of East Bronx low-service area, where a tremendous post-war expansion and building development had rendered the original small supply and distribution mains seriously inadequate in maintaining serviceable pressures under the low Croton gravity head. The need was apparent for a comprehensive hydraulic or gradient survey to determine the necessary size and location of needed trunk mains or other changes required, and such a survey was made in the latter part of 1924. (See Fig. 1.)

With the center of distribution about 4½ miles from the source and limited by the head of 133 ft. at the source, it was considered economically feasible to maintain the area below the 40-ft. contour on the Croton low or gravity service. This would allow a 24-ft. pressure loss in maintaining a minimum pressure of 30 lb. per sq. in. at the 40-ft. contour or highest elevation.

The hydrants selected for use in the survey were those at important points or intersections along trunk mains, main thoroughfares and centers of rapidly developing areas. The nozzle elevation of these hydrants was obtained by leveling. Incidentally, turning points for the levels were largely taken on nozzles of intervening hydrants, and these hydrant locations and elevations were recorded by routes for future use in the office or by field parties. The pressures were taken with two very accurate test gages, graduated in lb. per sq. in., 0.1 lb. per sq. in. being readily estimated. Pressures along the lines were taken at intervals of 2 000 to 3 000 ft. simultaneously with readings at the important intersections of the lines with their source of supply in other trunk mains. The reading recorded at each point was the average of 5 one-minute readings averaged mentally by the observer. The heads computed from these pressures, added to the elevations previously obtained gave the hydraulic gradients at these points. These gradients were then spotted on a skeleton map drawn to an approximate scale of 3 000 ft. to the inch. On this map was also indicated at various points the limit of fire supply. The data thus obtained presented on such a map give a very comprehensive "bird's-eye view" of existing conditions during hours of greatest demand, and excessive pressure losses along the various lines are conspicuously apparent. The loss between any two points obtained as described is dependably accurate for use in determining the size of the trunk main needed to correct unsatisfactory conditions.

In determining the location and size of main needed, other less tangible though important factors have to be estimated and used. The New York gradients were obtained in December, 1924. It was estimated that maxi-

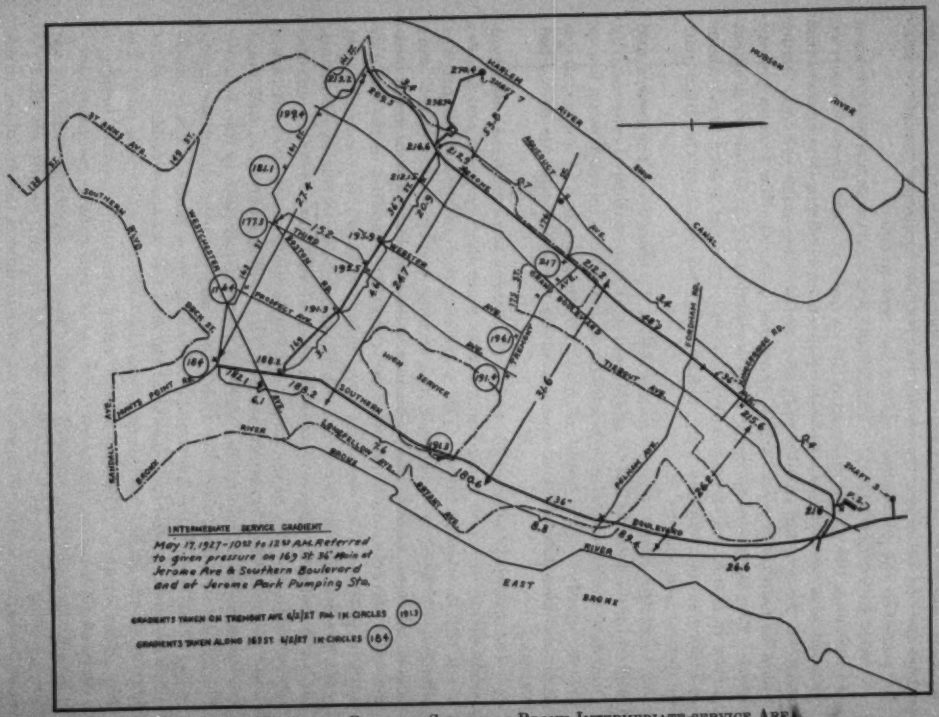

FIG. 2.— HYDRAULIC GRADIENT SURVEY OF BRONX INTERMEDIATE-SERVICE AREA

mum summer-time consumption in the area studied including large tent and bungalow colonies would be 25 per cent. greater than in December, giving pressure losses 50 per cent. higher than those indicated. Also it was assumed that at the rate of development then under way the consumption of the area might double in five to ten years. Thus the maximum consumption in the near future might reach 200 per cent. of that existing at the time and season of the survey.

The data on existing conditions, as presented on the map, clearly indicated, by gradient slope, the location and length of needed trunk mains. The size of main was determined by assuming an increased flow, corresponding to the estimated increase in consumption, and selecting a size sufficient to carry this flow with a loss of head reduced to an amount in accordance with good practice and safely within the allowable limit of the source of supply and the distribution system. Consideration was thus given not only to improvement of the pressure but to maintaining the improvement with reasonably distant future demands.

The recommendations for new mains, based on this survey, included the addition of 8 000 ft. of 36-in., 11 000 ft. of 30-in. and 19 000 ft. of 20-in. pipe. It was estimated that the larger sizes recommended would improve pressures by about 4 to 5 lb. The 20-in. pipe was mainly intended to strengthen the gridiron for fire supply and to so interconnect the two larger supply mains that either could serve alone in emergency.

Bronx Intermediate Service. Since introduction of the Catskill supply, the Bronx intermediate service has been greatly extended (Fig. 2). Two large north-and-south trunk mains of the original Croton system, one on the east and one on the west side have been utilized. The present main source of supply for this service, as previously mentioned, is from the Catskill shafts along the pressure tunnel lying west of the service and paralleling the original main arteries. The lack of trunk-main capacity in an easterly direction from the tunnel to the original main and to centers of distribution has been overcome by hand regulation at the shafts, increasing the daytime pressure and output at these points to that necessary to maintain serviceable pressures at the centers of distribution and reducing nighttime pressure at the source to avoid excessive pressures on the distribution system. The problem here presented did not involve the design of a system of reinforcement to provide serviceable pressures with a very limited head at the source, but rather the determination of size and location of mains necessary to so reduce the excessively varying pressure-losses as to permit maintenance of fairly uniform pressures on the system with a comparatively constant head at the tunnel. Such an arrangement is also desirable to permit more or less automatic regulation to replace hand regulation and to obviate unavoidably serious pressure-fluctations at the extremities of the system. In the solution of this problem a gradient survey was made using the same method as previously described. In this case, however, on account of the wide fluctuation due to high pressure losses, additional care had to be exercised

to have the gradients obtained under practically simultaneous and normal conditions that they might be comparable in all combinations on the final skeleton map. This required some repetition of work and averaging of results, but the variations observed were generally negligible in comparison with the pressure-losses determined.

Omitting details, the final map of results showed very flat gradients along the two original north-and-south trunk mains considered as "headers," but a serious daytime slope or pressure-loss between the pressure tunnel and the westerly 48-in. header amounting to 53 ft. along the 3 500 ft. of 36-in. feeder. An additional loss of 25 ft. in the 9 000-ft. continuation of this main to the easterly header was also observed. The map presented other relatively serious additional losses along secondary feeders so that, for example, at one point well within the service area the daytime head was 92 ft. or 40 lb. below that maintained on the supply main at the shaft. As before, the desirable location and necessary length of new mains was quite apparent on the map. The determination of size was made by selecting a pipe of such capacity, relative to existing mains, as to bring excessive pressure losses down to those corresponding to good practice and well-controlled pressures. The mains recommended included 4 000 ft. of 48-in. pipe to eliminate the excessive pressure loss between the tunnel and the westerly 48-in. header and 11 000 ft. of 36-in. pipe from this header to the easterly one to more nearly equalize the work performed by them.

Surveys of this kind establish the gradients available throughout the existing system, and, together with the determined elevation of hydrants, or more roughly the street-atlas elevations, are used daily on investigations of pressure troubles and in answering inquiries coming to the information desk as to pressures at specific locations.

Borough of Manhattan. In the borough of Manhattan a gradient survey of somewhat different character was made primarily for use in promptly solving these everyday important problems. Briefly, it consisted of a series of east-and-west gradient profiles or "hydraulic cross-sections" of Manhattan Island taken approximately ten blocks apart from the Battery to Spuyten Duyvil. It included about 300 hydrants on all of which pressures were taken during the business hours of one day. The pressures were taken with accurate gages that had been previously tested and calibrated, the corrections being applied to the gage readings in making computations. The nozzle elevations were determined by levels. The hydrant locations, elevations and gradients were listed by streets in geographical order and used daily by the parties on pressure investigations. For office use a street wall-map was prepared, in which the four service zones are shown in color. The gradients of the service at different general locations are indicated in large figures that can be read from the complaint and information desks.

These gradients were taken in September, 1923, under generally normal distribution and weather conditions and have been found very useful as standard reference gradients for solution of the varied problems involving

(1) depreciation of service due to increase of consumption (2) need of reinforcement and (3) the improvement secured by new installations or changes in distribution. They are useful in locating breaks in mains, open blow-offs, closed valves or restricted circulation and for intelligently advising a complainant as to the justification of his complaint. They enable us to tell him whether the trouble is local and under his own control, what pressure to expect on the street main, and whether new or larger plumbing will improve conditions and supply water to his top floor. They also point promptly to open service-boundaries and unwise changes in shaft distribution or regulation. The gradient network indicates very conspicuously the areas in need of better supply mains, and it has been used together with special gradient tests as a basis of selecting the sizes and routes of large trunk mains. Thus about 2 500 ft. of 30-in. steel and 4 500 ft. of 20-in. and 16-in. cast-iron main have been laid east and west from one Catskill shaft to flatten the gradient and to relieve the overload previously carried on the shaft next adjacent along the tunnel. There have been laid 4 350 ft. of 36-in. steel main to reduce a 16-ft. gradient slope across Central Park and improve pressures on high ground to the east. A contract is now under way for about 4 200 ft. of 48-in. and 3 800 ft. of 36-in. pipe from the 48-in. low-service aqueduct mains eastward to improve upper eastside low-service pressures by reducing the present gradient slope of 32 ft. in 5 000 ft. A contract is to be prepared for about 4 500 ft. of 30-in. main to reduce a present pressure loss of 25 to 30 ft. between the Catskill tunnel source and the extremities of the system, and to more fully utilize a Catskill shaft where output is now restricted.

Fire Supply. Fire supply presents an ever present problem in all distribution systems. If the area in question is large and the possible fire draft a small percentage of the total consumption, the solution of the problem probably does not involve consideration of trunk mains, but rather tests to locate points where possible fire draft would exceed the limited capacity of the local gridiron mains. If the consumption of the area is small in comparison with a possible fire draft, the solution may call for supply mains larger than necessary for the normal domestic and business consumption. In other words, the provision of suitable fire supply in such areas carries with it ample provision for daily consumption. In New York and other large cities the areas involving the greatest hazards are protected by an independent fire-supply system on which the pressure is raised at times of fire to suit the demand. Outside such protected areas of New York City, the solution of the problem of fire supply is based on tests made substantially as in other cities. A group of hydrants is selected for simultaneous discharge. Another hydrant on the side of the group opposite the apparent supply, or possibly within the group, is selected for observation of static and residual pressures. On signal from the man in charge, the observer at each discharge hydrant opens his hydrant fully and measures the discharge head by means of a Pitot tube attached to a low-reading pres-

sure gage. The discharge is obtained from curves of discharge for the various pressures and nozzle sizes. The observer at the pressure hydrant notes the static or normal pressure and the residual pressure with all hydrants open. The limit of capacity of the system at that location is estimated as the total discharge obtained from the group multiplied by the square root of the ratio of the static pressure to the drop in pressure observed with all hydrants open. If this limit of capacity falls below the requirements of, let us say, third or fourth alarm fires for that area, depending upon the hazard at the location, the final solution of the problem involves provision of mains whose size and location is usually obvious.

Occasionally the problem of safeguarding domestic and fire supply for an isolated high-service area may be solved by insertion of check-valves on the boundary between the high service and an adjacent low service of sufficient head to reach the high ground under the head as reduced by the emergency demand. Such valves, if not in occasional action, should be watched and frequently tested.

Pressure-regulating Valves. In the control of pressures in a system supplied by reduction of head at the source, the use of automatic pressure-regulating valves is frequently logical. Such valves are used on the New York system but not as extensively as originally intended. In general the nonuse is not a discredit to the valves but has been due to the unusual conditions imposed on their operation. The pilot valves of automatic regulators are actuated by variations in downstream pressure. In an overloaded supply main with a high friction loss between the source and the center of distribution the variations of demand in the system are not effectively transmitted as variations in pressures at the pilot valves. This is due to the fact that the pressure loss varies as the square of the flow or demand. A large increase in demand and a much larger pressure drop are therefore required to actuate the regulator at the source. A regulator placed midway on the line also cannot operate satisfactorily due to the highly varying head on the upstream side. The regulator, tending to maintain a constant difference of pressure, transmits these variations to the downstream side. Regulators working in parallel with near-by interconnections of their discharges have given unsatisfactory results because of the tendency for one of them to close as the other opens and *vice versa*. Where pressure has to be raised excessively at the source to maintain daytime pressures on the system the difficulty of utilizing regulating valves is obvious.

Underground Leakage. The control of underground leakage is a subject by itself too broad to be included in this miscellany. It is, however, an important problem, especially where subsurface conditions permit development of large and extensive leakage without surface indications. In New York City these conditions obtain in a very marked degree in the two boroughs of Manhattan and the Bronx. During years of plentiful supply we have with the small force of men available for the purpose been unable to do more than locate and repair leaks responsible for complaints of flow

into basements, electrical conduits, steam lines, subways and other subsurface structures.

In the regular course of events, such complaints are received at the complaint desk of the main office, and the order for investigation is telephoned to the repair shop of the district concerned. This order is followed by a copy of a written order, the original remaining at the office. Space is provided on the order blank for the foreman's report. When the leak is not found after a reasonable effort has been made to do so, the complaint is referred to the special waste-detection force and is given study under engineering supervision. Practically every case thus referred is different and requires original and serious engineering study on the ground to solve the problem and locate the leak, frequently a few blocks away, through a maze of subsurface conduits, gas and water mains, vaporizing steam lines, transit subways, and other structures.

It is anticipated that in or about 1935, New York City's consumption will have reached the capacity of its present sources of supply, and as this will probably precede the completion of an additional supply, conservation measures are now being considered and adopted. Systematic surveys to locate and repair underground leaks, many of which exist without causing complaint, are being studied. One or two field parties are now being used on surveys with the main object of determining how much underground leakage can be stopped, what the cost will be per million gal. saved, and what force and organization will be necessary to till this fruitful field in the few years remaining. Aside from the saving of water, leak-detecting surveys by small districts are a decided help in maintenance by incidentally locating and correcting various defects, such as closed valves, open service-boundaries, and restricted circulation. On the second of these experimental districts now under investigation an unexpected pressure drop occurring on the closing of the boundaries to isolate the district led to the location of a 6-in. blow-off wide open on a 20-in. main and discharging 6 m.g.d., as measured, into the sewer. This blow-off had not been indicated on distribution maps since about 1908, but there is evidence from which it may be assumed that the blow-off had not been open during that length of time. We do not expect such "finds" in all the trial districts to be investigated, but this case found in the second district investigated serves as a suggestion of one possible source of tremenduous unobserved waste in a system that includes hundreds of such blow-offs; but for the district waste-survey this leak would still be running.

Discussion.

In the discussion that followed this paper the following points were brought out: William W. Brush* called attention to the need for maintaining satisfactory pressures in the distribution system even when primary feeders had to be shut down. He stated that duplication of primary mains

*Chief Engineer, Bureau of Water Supply, New York, N. Y.

was justified by modern requirements, and gave it as his belief that a large city should have a trained engineer on duty at all times of day and night to cope with the distribution-system troubles that may arise.

A question relating to the use of pressure-regulating valves between high- and low-pressure systems was answered by Mr. Brush, W. C. Hawley* and John T. Robertson, who gave it as their experience that satisfactory results could be obtained with the Ross regulator valve set so as to maintain a certain minimum pressure on the low-pressure side. The notifying of householders or other water takers of the shutting off of street mains and the responsibility of the water department for the collapse of hot water tanks due to siphonage was considered by the Conference. Stephen H. Taylor† and David A. Heffernan‡ stated that no damages could be collected in Massachusetts from the water department unless negligence could be proved. The rules and regulations of the department protected it adequately. It was the consensus of those taking part in the discussion that consumers should be notified whenever a shut-off was planned, but that repair of breaks in mains requiring their being placed out of service suddenly could not await the notification of the consumers affected. Large water takers, however, are commonly advised of the shut-off as soon as possible. Vacuum-relief valves, although much improved in recent years, were held to be somewhat unreliable, and the opening of a hot-water faucet was advocated whenever mains are to be shut off or water meters are to be replaced. Education of water takers in what to do when the water is shut off, and how to take care of their plumbing system was considered worth while by E. C. Hopkins.§ The use of a low-pressure alarm system for industrial water takers was not believed to be satisfactory by H. O. Lacount¶ because the alarm would have to be set at too low a pressure and because it was a mechanical contrivance and as such could not be relied upon at all times. The practice of notifying the fire department of shut-offs seemed to vary in different communities.

The use of two independent connections protected by check-valves was considered advisable to insure a supply to factories during shut-offs. Installation of check-valves on services to dwellings equipped with pressure-type hot-water boilers was held to be dangerous practice. It seemed the consensus of the Conference that while these check-valves prevent the backing-up of hot water into mains, and the collapse of boilers by siphonage, they constitute a grave danger when boilers become overheated and pressure-relief valves fail to function. Check-valves were also stated to reduce appreciably the pressure on service connections.

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§President, The Crystal Water Co., Danielson, Conn.
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ELECTROLYSIS INVESTIGATIONS.*

BY C. F. MEYERHERM.†

[*Read September 19, 1928.*]

Electrolysis surveys and electrolysis mitigation are subjects that in many cases receive little or no consideration until a failure occurs or serious corrosion is discovered on some part of the underground piping system. Then there is a flurry of excitement, some form of mitigation is attempted, usually without much consideration of the general aspects of the problem, and the agitation subsides until there is a subsequent recurrence of trouble.

As a contrast, it is interesting to note that one of the largest operators of underground lead-sheathed cable-systems in this country, after careful analysis and study of all the factors involved, has found that an expenditure for electrolysis testing and mitigation of three-tenths of a cent per year per dollar invested in underground plant was more than justified from an operating standpoint. If a cable-owning company, whose electrolysis problems are comparatively simple, finds a comprehensive electrolysis test and mitigation program profitable, it should prove correspondingly more advantageous from an operating standpoint to a pipe-owning utility. An underground piping network is not only more accessible to stray current than a cable system, but failure also involves more expensive repairs and in addition serious fire, explosion, sanitary or flood hazards.

Many operators of water and gas systems accept the absence of self-evident failures by electrolysis as *prima facie* evidence that no electrolysis is taking place. This is a misleading assumption since anyone who has had actual experience with electrolysis problems knows that stray-current corrosion occurs at comparatively slow rates over extensive sections of the underground network, while failures occur only at scattered spots and sometimes do not develop until long after the mechanical strength of the affected structure has been seriously impaired or destroyed. Such failures often take place at most inopportune times. Engineering and operating considerations, such as those of public safety, continuity of service and the maintenance of favorable public relations, all demand foresight in the treatment of the electrolysis problem; and foresight, to be effective, must be based upon an engineering knowledge of facts.

A properly planned and effectively directed electrolysis-control program should prove economically profitable from an insurance standpoint to any utility operating an underground metallic structure subject to stray-current action. Such a program should not only detect bad situations and make it possible to forestall actual failures by pre-emergency repairs, but it should also arouse and maintain active general interest in matters of

*A paper presented at the Superintendents' Conference.
†President, Albert F. Gans, Inc., New York, N. Y.

FIG. 1. — MOBILE TESTING EQUIPMENT FOR ELECTROLYSIS SURVEYS.

electrolysis on the part of all the utilities operating underground systems. Such an interest is one of the best incentives toward progressive and constructive improvement in the general electrolysis situation, and it is a practical assurance against prolonged continuance of dangerous conditions.

A program of electrolysis control necessarily involves some form of electrolysis survey, and such surveys vary as much in their scope and character as the situations to which they are applied, the objects sought, and the engineers directing the work. A routine survey conducted independently by a single utility can be made with a small test force and limited indicating-meter equipment, but it is extremely confined in its adaptability and usefulness. With present-day railway-operating practices and the complexities of subsurface systems, it becomes necessary almost always to make a more comprehensive study of the situation.

Practically every present-day electrolysis problem involves a multiplicity of underground structures, and in most cases several pipe, railway, or cable systems exist and are more or less closely interrelated. Under these conditions a survey to obtain complete information regarding the electrolytic and general electrical relations of all the underground systems must include tests on all these systems. This not only necessitates a large and varied assortment of indicating and recording instruments and special test apparatus, but also an experienced personnel and the effective coöperation of all the utilities operating underground systems. A large number of instruments of both the recording and indicating type are necessary in order to obtain the required simultaneous readings and to make the most economical use of the test personnel. Special equipment, tools, wire reels, contact devices and automobile transportation are essential to systematic and rapid progress. To give some idea of the nature, variety and amount of apparatus needed for the carrying on of an extensive electrolysis survey, there is shown in Figure 1 the specially equipped electrolysis truck such as was used in recent tests in Chicago, Winnipeg, Cleveland, Youngstown, Louisville, New Haven, and other cities. With this equipment, large or small jobs can be handled in a minimum time and at minimum expense, while comprehensive data are secured.

The electrolysis survey or program, to be effective, should be under the more or less continuous supervision and direction of an engineer who has had considerable experience with practical electrolysis problems and who can give a considerable part of his time directly to the work. Too many electrolysis surveys consist merely of a mass of unrelated or uncorrelated readings taken by men who have no conception of the objects sought or of the significance or lack of significance of the data they are taking. Experienced direction of the field work is essential. Test men will often conscientiously follow a predetermined schedule and entirely miss important side issues bearing on the general problem, or they will be misled by interesting but inconsequential facts. An electrolysis survey to be of real value must be more than a mere compilation of figures and charts. It must be a true

picture of existing conditions, and such a picture can be presented only by an engineer with broad, general electrolysis experience, an accurate knowledge of actual field conditions and an understanding of special local factors pertaining to the job in hand.

Discussion.

In opening the discussion of this paper, the Chairman of the Conference, Stephen H. Taylor* read the following letter written to him by Mr. Briggs, Superintendent of Wires, New Bedford, Mass.

"In regard to our recent conversation about Leadite joints in water mains: Leadite is regarded as non-conductive, or, at least, a poor conductor of electricity. Therefore, when Leadite is used to make joints in water mains you insulate each section of your iron pipe.

In all large cities where such large amounts of electricity are used, you will find stray currents of electricity flowing in underground iron piping, and when the current is present in large enough volume where it leaves the pipe, is the point where the damage is caused. Now the question arises, would it be better to have our underground piping a continuous metal conductor or broken up into sections as we do when we use Leadite for joints? Let us say we have electrolysis; what happens? With a continuous metallic piping system, the current will flow to some point where it can leave the piping system. In most cases there will be one or two points where the current can leave and at these points you will receive the damage. With the Leadite joint piping, with the same conditions arising, the current is trying to get from one section to another, and you will have your electrolysis damage occurring at each point.

To make your underground piping a continuous metallic conductor, drive three lead wedges in each joint before you pour your Leadite. By the use of these lead wedges, making your water mains a continuous metal conductor, you will localize the points where your current will flow from your mains and be able to probably take care of this by installing at the points a good metal conductor which will allow this current to flow from this secondary conductor and then leave without damage to this installed device.

With Leadite joints, as stated above, if you have a volume of current flowing you will get electrolysis damage at each joint, thereby weakening your whole system, rather than in the case of the continuous metallic conductor piping you localize it at one or two points. Also from the standpoint of tests, you will more readily discover and locate the points of damage than with the Leadite joint system."

The Chairman then pointed out that according to Technologic Paper No. 52, of the U. S. Bureau of Standards "Electrolysis and its Mitigation," the resistance of Leadite joints decreases to about 1/2000 of its initial value in four months and to 1/19000 in four years.

Mr. Briggs' suggestion to decrease the resistance of water pipes laid with Leadite joints was opposed by Charles W. Sherman,† Roswell W.

*Superintendent of Water Works, New Bedford, Mass.
†Of Metcalf and Eddy, Consulting Engineers, Boston.

Roper[1], John D. Capron[2], W. C. Hawley[3], and D. W. Barron[4] for the reason that Leadite joints remain relatively resistant even with age, and that decreased resistance will bring with it increased current flow and, therefore, increased corrosion at points where the current leaves the pipe. The danger of increased electrolysis at isolated Leadite joints in lead-joint pipes, where connections between old and new mains are made by crosses for example, was mentioned by Mr. Barron, who pointed out the advantage of bonding the pipes at the Leadite joint in such cases.

The importance of good rail-bonding of electric railways was called to the attention of the Conference together with the use of return cables where pipes and other conduits are endangered by electrolysis. Intermediate bonding of pipes and rails for the "drainage" of electricity from the pipe lines was considered advantageous by Messrs. Roper and Barron under certain conditions.

The grounding of electric circuits, light, telephone and radio was discussed at length by the Conference. It was brought out that, although the current flow is supposed to be small, currents sufficient to produce "shocks" in changing water meters have been a nuisance in the experience of David A. Heffernan[5], W. C. Hawley, and others. It seemed the consensus of those taking part in the discussion that the ground should be made on the street-side of the meter, but that this might still cause objections in working on service lines in the street. Interference of an electric circuit grounded to water pipes, until disconnected, was mentioned by C. J. Clogston.[6] The practice of grounding electric circuits to water pipes was defended by Walter P. Schwabe[7] and C. Stephen[8] in the interest of the protection against electric shock and fire.

It seemed the consensus of the Conference that the Committee on Grounding of Currents on House Plumbing appointed by the NEW ENGLAND WATER WORKS ASSOCIATION to act with a similar committee of the American Water Works Association should be encouraged to render a report as soon as practicable and that joint consideration of electrolysis problems with other interested utilities was desirable.

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[1]Engineer, Board of Water Commissioners, East Orange, N. J.
[2]Research Engineer, U. S. Cast-iron Pipe and Foundry Co., Burlington, N. J.
[3]Chief Engineer, Pennsylvania Water Co., Wilkinsburg, Pa.
[4]Of Albert F. Ganz, Inc., New York, N. Y.
[5]Superintendent of Water Works, Milton, Mass.
[6]Superintendent of Water Works, Barton, Vt.
[7]President, The Northern Connecticut Power Co., Thompsonville, Conn.
[8]Montreal.

COPPER AND BRASS WATER SERVICES.*

BY WILLIAM G. SCHNEIDER.†

[*Read September 19, 1928.*]

Every user of pipe for domestic water purposes should ask himself certain questions before he selects the pipe material. These questions may be set as follows:

1. Will the pipe impair the hygienic quality of the water?
2. Will the pipe corrode rapidly?
3. Is the pipe easily installed?
4. Is the pipe economical?
5. Will the pipe affect the water supply either by rusting or becoming clogged with rust so that it will be a source of annoyance and complaint?

For copper and brass, when used for services or for plumbing pipe for handling domestic drinking water, the author should like to answer these questions as follows:

1. *Will Copper and Brass Pipe Impair the Hygienic Quality of the Water?*

Leading authorities have stated that no danger exists nor need any anxiety be felt regarding any possible effect of copper or brass on domestic drinking water. Many thousand pipes of these materials are in daily use and are entirely satisfactory.

2. *Will Copper or Brass Pipe Corrode Rapidly?*

A proper installation of copper or brass pipe will give satisfactory service. Copper and brass are highly resistant to corrosion. Copper pipe 5 400 years old that was used for conveying water in Egypt is still in good condition and serviceable. Within modern times an installation of brass pipe in the Parker House in Boston is of interest. Installed in 1856 it was found to be in excellent condition and suitable for reinstallation when this famous hostelry was torn down nearly seventy years later. Considered for service pipe where they have to withstand not only the corrosive action of the water on the inside of the pipe, but in addition the corrosion caused by the soil on the outside of the pipe, copper and brass have proved themselves highly satisfactory and are being used for services in increasing quantity.

3. *Are Copper and Brass Pipe or Tubing Easily Installed?*

Standard-size copper or brass pipe is as easily installed as galvanized iron. Standard brass fittings are on the market and are carried by jobbers and plumbing supply houses to the same extent as fittings for standard iron or steel pipe. Brass pipe is just as easily threaded as any other piping.

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* A paper presented at the Superintendents' Conference of September 19, 1928.
†Copper and Brass Research Association, New York, N. Y.

Copper tubing for use in services with compression fittings presents a cheap and satisfactory method of making the connection to the main. A separate gooseneck is not required; the corporation cock, if of the compression type, will hold the flanged copper tubing. No thread cutting is necessary and a hack saw suffices for cutting the copper tubing to the requisite length. With standard-size brass or copper pipe the gooseneck may be made of the same material as the pipe providing it is soft annealed.

4. *Are Brass and Copper Pipe and Tubing Economical?*

The initial cost of a service-pipe installation is made up of the following items:

1. Opening up and closing the ground or forcing the pipe through the ground.
2. Cost of the pipe.
3. Cost of connecting the pipe.

The first item is the same for all the materials mentioned and regularly used for services.

The initial cost of various materials per foot is about as follows:

	¾-in.	1-in.	1¼-in.
Galvanized steel pipe	$0.07	$0.104	$0.14
Galvanized wrought-iron pipe	0.125	0.18	0.245
Brass pipe, standard-pipe size	0.32	.046	0.67
Copper pipe and deoxidized copper pipe, standard-pipe size	0.32	.046	0.671
Copper and deoxidized copper tubing for services and use with compression fittings	0.18	0.23	0.28
Lead pipe, 2A	0.30	0.41	0.49

As far as the cost of connecting the various materials is concerned, the author believes that the different materials array themselves in the following order:

1. Copper tubing using compression fittings.
2. Brass or copper standard-size pipe and galvanized-steel and wrought-iron pipe.
3. Lead pipe.

Additional economies may be effected by using copper tubing or copper and brass pipe because it is not necessary to use the same diameter as when the pipe is made of iron or steel. Where corrodible materials are used, it is necessary to make a proper allowance for rust clogging, which cannot take place where copper or brass is used. The relative savings in pipe sizes which are made possible by the use of brass or copper for services or for plumbing purposes are:

In hot-water lines, ½-in. copper or brass pipe will do the work of 1-in. iron pipe and the other sizes will be related about as follows: ¾-in. and 1¼-in.; 1-in. and 1½-in.; 1½-in. and 2½-in.; 2-in. and 3-in.

In cold-water lines the relation will be ½-in. copper or brass and ¾-in. iron; ¾-in. and 1-in.; 1-in. and 1¼-in.; 1¼-in. and 2-in.; 2-in. and 2½-in.

5. Will Copper or Brass Pipe or Tubing Affect the Water Supply, Either by Rusting or Becoming Clogged with Rust so that it will be a Source of Annoyance and Complaint?

The data as to the cost of service pipes cover only the initial costs. Where the material used is highly resistant to corrosion and very little or not at all affected by the water or soil, the first cost is the last cost. But where the material is eaten away by rust, it is only a matter of time before it will have to be renewed. Under such conditions it is entirely possible that many replacements of the corrodible material will have to be made during the lifetime of the non-corrodible service. As replacement practically means the installation of a new service and as the cost of the material is but a small part of the total cost of the installation, it is advisable to use at the very beginning that material which is most satisfactory.

General Data on Copper and Brass Tubing and Pipe.

Cold-drawn copper tubing or pipe develops a tensile strength of about 40 000 lb. per sq. in. with an elongation of about 10 per cent. in 4 in. Soft annealed copper tubing or pipe develops a tensile strength of about 30 000 lb. per sq. in. with an elongation of about 25 per cent. in 4 in.

The sizes of copper tubing used for services with compression fittings are as follows:

Nominal Size. in.	Outside Diameter. in.	Gage. in.	Weight. lb. per ft.	Test Pressure. lb. per sq. in.
$\frac{1}{2}$	$\frac{1}{2}$	.049	.343	1 140
$\frac{3}{4}$	$\frac{3}{4}$	.065	.640	1 040
1	1$\frac{1}{8}$	.065	.838	780
1$\frac{1}{4}$	1$\frac{3}{8}$	.065	1.04	630
1$\frac{1}{2}$	1$\frac{5}{8}$	.072	1.36	580
2	2$\frac{1}{8}$	.083	2.06	520

Straight lengths of copper tubing for services may be obtained up to 35 ft., and coils from 40 to 60 ft.

The copper tubing in comparison with the same length of standard-size pipe delivers the respective quantities of water shown below:

Tubing. in.	Standard Pipe. in.	Length ft.	Flow in Tubing.
$\frac{1}{2}$	$\frac{1}{2}$	20	.7% less than in standard pipe.
$\frac{3}{4}$	$\frac{3}{4}$	20	2.2% more than in standard pipe.
1	1	10 to 100	5% to 15% more than in standard pipe.

To test the bursting pressure of copper service-pipe, sections of pipe were filled with water. One end of the pipe was closed with a plug and a plunger inserted in the other. Pressure on the plunger was exerted by a regular testing machine until the specimens burst, with the following results:

BURSTING PRESSURE — LB. PER SQ. IN.

Size, in.	Copper Service Pipe.	Iron Pipe.	Lead Pipe.	
			Extra Strong.	Double-Extra-Strong.
¾	6 120	1 140	1 700	2 600
¾	4 815	1 000	1 500	1 600
1	3 890	920	1 100	1 200

No leaks at joints were noticed with copper tubing and compression fittings.

To test the resistance of copper services to bursting due to freezing, the pipes were tested by allowing water to freeze in them. The water-filled copper service tube after twelve hours at a temperature below zero showed an increase in diameter of about 12 per cent. After thawing the pipe and again filling with water and holding at below zero for three hours a further increase in diameter was noted of about 5 per cent. A third test showed a further increase in diameter of about 2 per cent. The double-extra-strong lead pipe increased slightly in diameter after the first freezing but became very flaky, showing a serious disintegration. Standard galvanized-iron pipe burst during the first freezing.

DISCUSSION.

In the discussion that followed this paper Edwin H. Arnold, G. E. Hawkins, C. W. Hawley, E. Le Royer, Marshall B. Palmer, and George F. Wieghardt reported the successful use of copper service pipes over periods ranging from 1½ to 7 years. Mr. Hawkins stated that copper pipe was cheaper than any other material and Mr. Hawley that he had found copper services cheaper than the lead services formerly employed in Wilkinsburg, Pa. C. W. Hawley and David A. Heffernan pointed out, however, that while copper pipe seemed to meet the demands placed upon it, the choice of material should be varied with local conditions, especially water quality. Instances of the failure of brass plumbing pipes were cited by them as well as by others. David A. Heffernan and Joseph A. Hoy brought out with what success cement-lined services had been employed at Milton and Worcester respectively.

Mr. Schneider, in answer to questions, stated that a proper installation using the right mixture of copper and zinc would prove satisfactory, also that he believed that annealed or semi-annealed brass pipe would stand three freezings satisfactorily, and that the amount of chlorine in chlorinated water supplies was not sufficient to affect copper and brass pipe.

SUPERINTENDENTS' CONFERENCE.

A Report of the Proceedings of the Superintendents' Conference of September 19, 1928.

Stephen H. Taylor,* Chairman.

Revenue from Fire Services and Unused Domestic Services.

W. C. Hawley.† Our utility commissions pretty generally have decided that the water utilities should own the service line from the main to the curb and the water meter. That means an investment of $25 to $50 per service, depending upon the cost of the service line for each individual supply. With a thousand consumers $25 000 to $50 000 would therefore be invested; with 10 000 consumers, $250 000 to $500 000. When the service is idle the water utility receives no benefit. The number of idle services is often surprising and may constitute perhaps 5 per cent. of the properties.

Why should not the water utility be able to charge a portion of the minimum rate or a portion of the service charge during the time the water is shut off? I realize that there are difficulties. On what basis should collections be made, especially if the property has changed hands while the water was shut off? In these days when we are trying to segregate costs of service and make our rate schedule cover those costs, it seems as though this point should receive attention in improving schedules.

In some of the summer or winter resorts, where properties are only occupied for two to four months during the year, it has been customary to charge half the minimum rate or half the flat charge while the property was vacant.

H. N. Blunt.‡ In a great many towns where the highways are paved the water utilities are notified that before the street is paved the water services shall be laid to every lot, involving the expenditure of thousands of dollars for which the utility receives no return. It has always been interesting to me to know how this question could be worked out. Under these conditions, it seems to me, that it is impossible to receive compensation even under the broadest interpretation of public service rulings.

W. C. Hawley. The Pennsylvania Utility Commission and others have refused to order service lines laid ahead of time.

H. N. Blunt. The Service Commission may not give the order but the local authorities will.

W. C. Hawley. We pay no attention to local orders. In one case

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†Chief Engineer, Pennsylvania Water Co., Wilkinsburg, Pa.
‡Superintendent, Palmer Water Co., Palmerton, Pa.

I told the Council that we were willing to put in service lines for anyone who would pay the actual cost. The Council decided that it would not ask the owners to pay and that it would advance the money itself. About sixty locations for service lines were selected. Forty of the lines laid were gradually put into service. Only one of them was of the right size and in the right location.

CALEB M. SAVILLE.* At Hartford we collect the minimum charge whether the service is used or not on the basis that the service is of value to the consumer. If the owner goes away for the summer, for instance, he wants to turn the water on when he returns. We often lay a connection to a pasture. While the pasture will not be used during much of the year water is wanted when needed. This "readiness to serve" is worth something. We read our meters every two months. We have to read them whether they are used or not. There are the billing and the clerical work. So that there seems to be no reason why there should not be a minimum charge. The minimum charge for water is low enough in any case.

As to the installation of pipes before the pavement is placed, our practice and experience has been very similar to that of Mr. Hawley. Years ago there were numbers of services put in on several streets. Very few of them have been used; yet the streets are now built up. Services were put in at each lot but the houses were never built at the right places. In one case the district developed into a high-class residential district; the architects, as a result, did not design the houses to have the water pipes coming in where the services were originally located. In another case, I believe, 1-in. services were put in, and the district, developing into a business section requires from 2- up to 6-in. connections.

As to the matter of charging for fire services supplying factories, this seems to me to be a local matter. At Hartford we make no charge for fire protection to mills, and very few mills have meters. If the service is large enough, as it is in most of the larger cities, to furnish a good fire supply, I believe that the citizens derive enough benefit from adequate fire service to the mills, from continuity of service and by keeping fire from starting in the mills, to warrant the municipal water department giving free water service.

CHARLES W. MOWRY.† The question of charging for water services for fire protection comes up periodically in the National Fire Protection Association and in the American Water Works Association. It always seemed to me that it is only fair for a water company to charge for the cost of installing fire services and for maintaining or inspecting the service.

CALEB M. SAVILLE. I agree to that.

CHARLES W. MOWRY. It has seemed unfair, however, to charge for the water used in putting out a fire. It is just as unfair to do that as to charge for the water used in putting out a fire in a residence, for example.

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†Associated Factory Mutual Fire Insurance Companies, Boston, Mass.

As a matter of fact, the property owner who has installed his own piping and sprinkler system makes less demand upon the water department than may his neighbor who has an old wooden fire trap and who has no fire protection at all. Therefore, it seems to me equitable to distribute the cost of fire protection in the taxes, as is usually done. To make the property owner pay an additional service charge for his fire service smacks of double taxation and discrimination.

THE CHAIRMAN. I, too, feel if, as in New Bedford, the expense of installing and maintaining the fire service is carried by the mill, that is about all that can be expected, inasmuch as it is a part of the city's duty, in the case of municipally-owned plants at least, to furnish water for fire protection.

W. J. WILSON. Owners reduce their insurance by 60 or 70 per cent. by putting in a sprinkler system. To render service for sprinklers the utility must provide mains that are large enough. Since the property owner saves 60 or 70 per cent. on his insurance, why should not the utility have the privilege of charging for the increased mains required?

CHARLES W. MOWRY. If the utility lays a large main to a factory, there is no reason why the owner should not pay for the main.

THEODORE L. BRISTOL.* May I ask Mr. Mowry if he has any objection to charging for fire services according to the size of meter?

CHARLES W. MOWRY. I feel that if any water is drawn from the system for industrial use it should be metered and the owner should pay for it. When the system is entirely for sprinklers, however, and there is no draft except for fires, a meter is unnecessary because there is no revenue from the meter. Moreover, water is only used at the time of a fire, and the owner should not be charged for it then.

THEODORE L. BRISTOL. Our experience shows that in many cases we get a large revenue from fire meters either through leaks in the yard or the use of water which would be stolen if it was not metered.

W. C. HAWLEY. We have had at least two cases where a detectometer in fire lines was very beneficial in the way of locating leaks. This is of value to the mill owners and to the Fire Underwriters. The great majority of mill owners is made up of honest men who will not tolerate the stealing of water. But water is stolen. A detectometer, or other instrument like it, is the only means of controlling private fire protection.

A. U. SANDERSON. If I recollect correctly the courts have upheld in certain cases the right of the public utility to collect a rate for a ready-to-serve charge, whether the water is used or not, for both sprinkler systems and private services.

THE CHAIRMAN. Does that ruling also apply to services to dwellings that are shut off?

A. U. SANDERSON. It does not, as far as I know.

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*President, Ansonia Water Co., Ansonia, Conn.

ROGER W. ESTY.* May I ask Mr. Mowry whether the dry-pipe sprinkler system is better than the wet-pipe system?

CHARLES W. MOWRY. The wet-pipe system is the most efficient because the moment the sprinkler opens the water is instantly delivered. In a dry-pipe system, filled with air under pressure, the air must first be entirely exhausted, or, at least, reduced down to about 30 lb. per sq. in. pressure, before the valve trips. This introduces a delay of from 10 seconds to two or three minutes, depending on the size of the system. In addition, there is the blast of air from these sprinklers, which in some cases will stir up the fire and make it far worse than it would otherwise be. In an unheated building of course there must be no water in the pipes; then the dry-pipe system is used.

FRANKLIN HENSHAW.† To prevent water being used from sprinkler systems illegitimately we install an electric alarm which rings a big bell the moment the pressure drops.

THE CHAIRMAN. Where is your alarm located; in the building or at the street line?

MR. HENSHAW. The alarm is located just inside the building wall, and the bell can be heard both inside the building and on the street.

ROSWELL ROPER.‡ One of the primary objects with those of us who use meters is to detect the smaller leaks that run day and night and waste much water in the long run. I feel very sure, after investigating the electrical system and the differential in pressure necessary to ring the bell that it allows enough leeway so that considerable slow leakage may exist.

THE EFFECT OF FLUSHOMETER VALVES UPON WATER-WORKS PRACTICE.

GEORGE A. KING.§ It seems to me that the flushometer valve is a serious menace to our water systems, for two reasons: the danger of water hammer, and the danger, if the valve is not properly constructed, from siphoning pollution from the closet back into the water pipe if conditions happen to be right.

DAVID A. HEFFERNAN. There was originally a little danger of siphoning by flush valves, but that has been eliminated. There is no trouble when the flushometer valve is set high enough.

There is a tendency today in installing plumbing fixtures to eliminate many things including air chambers. The only way to avoid water hammer is to put an air chamber at some point on the house side where it will do most good. Flushometers are used either on the direct system or on the tank system. In large residences I think it is the best plan to put in a tank, and in that way plenty of water may be obtained to supply the fixtures. One cannot use an ordinary service pipe,—say ¾- or 1-in.,—and expect to

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§Superintendent of Water Works, Taunton, Mass.

tie it up to a flushometer; there will be insufficient water. There is a way to overcome this difficulty by putting in a good pressure tank.

ROGER W. ESTY. I know of several cases where the use of a tank has made it possible to operate flushometers that would not function properly.

We have had some experience with water hammer due to flush valves. In one case this amounted to about 40 or 50 lb. per sq. in. But there are flush valves that do not create water hammer. By extending a pipe up beyond the flush valve water hammer is practically eliminated.

I do not believe it necessary for any department with ordinary house service to increase the size of the service pipe or meter just for the use of a flushometer when satisfactory operation can be secured by a small expenditure of about $10 for a tank.

GEORGE F. MERRILL.* Last winter, a consumer wanted to install a flushometer. After the plumber had extended a 1½-in. pipe to the fixtures, the flushometer would not work. I recommended that a compression tank be installed. A small boiler was used for this purpose, and there was no trouble after that. The compression tank is a good thing because it absorbs water hammer and provides a good air cushion on a flushometer service. We had to increase the size of the meter from ⅝- to ¾-in. in order to get sufficient volume of flow.

CHARLES J. CRUMP.† When flushometers on a 2-in. supply which had been in service for about 20 years failed to work, I advised using a 30-gal. tank. Now two flushometers are working and there is plenty of water.

GEORGE F. WEIGHARDT. In Baltimore, Maryland, we had considerable difficulty in providing enough flow for flushometers with a normal average pressure of about 30 or 40 lb. Eventually we established the rule not to allow flushometers in buildings unless they were provided with a 1-in. service. That solved the problem.

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*Ware, Mass.
†Superintendent of Water Works, Peabody, Mass.

SPECIFICATIONS FOR FIRE HYDRANTS AND VALVES, WITH SPECIAL REFERENCE TO VALVE STEMS.

A Report of the General Discussion of September 20, 1928.

ARTHUR D. WESTON,* Chairman.

FRANK A. MCINNES.† I take it that the specifications referred to as possibly requiring revision are those published by the American Water Works Association in 1916. As the question is particularly in reference to valve stems, I will confine myself to this subject.

The specifications referred to simply require that the stems shall be made of solid brass or suitable non-corrodible metal, free from defects, and shall have a tensile strength of not less than 45 000 lb. per sq. in.

Apart entirely from the question of the suitability of solid brass for stem material, there is not a word about yield point, the relation of yield point to ultimate strength, elongation, etc. To recognize tensile strength only, without provision for the necessary ductility surely leaves much to be desired in a specification for valve stems.

The fact that a notable advance has been made since 1916 both in the alloys used in stems and in the methods of manufacture of the metal is alone a sufficient reason for a move in the line of new specifications.

It must be kept in mind that a stem can be too strong, as it is when capable of putting the gate in which it operates out of commission before the stem itself fails. The Boston Water Department has used for many years gun metal for stems; this alloy of 88 per cent. copper, 10 per cent. tin and 2 per cent. zinc, with a tensile strength of about 30 000 lb. per sq. in. and an elongation of 14 per cent. in 2 in., has proved entirely satisfactory when first-class materials are used and the casting is properly done.

It is desirable and informing to have an interesting discussion, but we must look ahead, see clearly the road on which we travel and the results we hope to attain. It would be a mistake for our Association to prepare a new specification for hydrants and valves. There is one agency only capable of getting the desired result in a final and satisfactory manner, viz., the American Engineering Standards Committee; if this body can be convinced a real demand for such a specification exists, it will appoint a Sectional Committee representative of the interested parties throughout the country to complete the project.

The more spirited and worth while our discussion is at this time, the better will the need of new specifications be shown.

H. O. LACOUNT.‡ The stem and the stem nut are certainly important

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†Consulting Engineer, Boston, Mass.
‡Associated Factory Mutual Fire Insurance Companies, Boston, Mass.

parts of a valve, and in our experience, it seems to me that the stem and the stem nut have been at the bottom of our troubles pretty largely. This refers principally to valves in private fire systems.

Now, up to some 18 or 20 years ago, I think the valves which were used in private fire systems were practically the same as those that were used in water-works practice generally, — the commercial valve with possibly slight modifications here and there to suit different requirements. We were then having reported to us through our Inspection Department many troubles due to crippled stems and broken nuts — so much so that we looked into the matter and made a study of the question of a more reliable construction. This led to specifications calling for a better grade of valve. That work was done in coöperation with a special committee of the National Fire Protection Association, on which we were represented. That Committee also had representation from the Underwriters Laboratories and the manufacturers — I think there was a Manufacturers' Association. Anyway, the result of that work was what was known as the National Standard Specifications. In the valve factories these uniform requirements resulted in an additional line of valves which the manufacturers referred to as the Underwriter valves, evidently because the insurance interests were largely responsible for the specifications. These National Standard specifications were approved by the Factory Mutual Companies, the National Fire Protection Association and the Underwriters Laboratories, and were satisfactory to the valve manufacturers and received their backing. So much for a little of the history leading up to and resulting in the National Standard specifications.

Regarding the particular point that Mr. McInnes referred to, special consideration was given to the stem of the valve and the stem nut. I think the American Water Works Association specifications for stems called for composition or other non-corrodible metal having a tensile strength of at least 45 000 lb. per sq. in. They also specified the stem diameters for different sizes of valves.

In our study, we finally decided that something further was needed and about 1921 there was inserted into the specification an article calling for a test for twist; in other words, the twisting moment, measured in foot-pounds, for the stem as assembled in the valve. In other words, just tensile strength and diameter do not necessarily give a satisfactory stem, because there are other factors, such as the pitch of thread, distance between supports and so forth, which enter and have a real bearing upon the final performance of the stem itself. But if we test the stem as designed and as assembled in the valve, we will include all these factors. This was worked out in a table calling for definite foot-pound twisting moments of the stems as assembled in the different sizes of valves which were submitted for examination. I do not mean that these tests were made on every individual valve in the shop. They were done for approval of the design and for checking purposes using thoroughly representative samples from stock.

For example, a 6-in. valve is required to withstand a twisting moment of 300 ft.-lb. You can get that any way you please, say by a 5-ft. lever with a 60-lb. pull. In the laboratory we use a good grade of spring scales. These are not scientifically accurate but they answer the purpose for a practical test, and the valves that are approved under the specifications will withstand that twisting moment.

I also remember that the 8-in. valve calls for a twisting moment of 400 ft.-lb. which is quite a severe test. Although our experience with these approved valves is perhaps still somewhat limited — we have received very, very few complaints of crippled stems made according to these specifications since the valves got into field use.

Coupled with this question I want to refer to the stem nut, because we did have a good deal of trouble with the lugs or the flanges of the nut which hold the disc to the stem breaking off and dropping the disc into the waterway. This difficulty was much helped by a simple device which any of us could think of, namely, by casting a web from the threaded section to the lug. That very materially strengthened it, and we have had much less trouble from that cause since.

Thus for our contribution to this discussion we just tear this leaf out of our own notebook of experience. With regard to these two points we think the specifications have really helped the performance in the field.

Personally I do not see any great difference between conditions as we meet them in the fire lines on private property and in the line that is in the street. These valves are all water valves, and for either service a better valve would result in less repair, saving the cost of repair, and saving the shutdown of the line which may be of vastly more importance to the public in general than just the repairs to a valve. Incidentally, the more reliable the device controlling any water line, the more it is likely in an emergency to meet the test, and the less somebody suffers.

Three specifications come to my mind; the American Water Works and the so-called New York City specifications, with which I am less familiar, and these National Standard specifications. To standardize, to bring all these together, resulting in one uniform specification, certainly is in line with the spirit of the day, which calls for standardizing all these various appliances and devices. If it should result in a somewhat better valve, it might justify a little more cost of the valve. I understand this so-called Underwriter equipment does cost slightly more than the other, although this undoubtedly could be reduced if factories could discontinue making two lines of valves for practically the same purpose.

My thought, in parallel with Mr. McInnes', therefore, is that the New England Water Works Association, either now or at a later meeting, might consider starting a movement towards that end, using the machinery which the American Engineering Standards Committee has available. I think such a discussion as we will have today and later may bring to the surface a little more definitely just what the field conditions are, and show

whether there is a need for more definite and perhaps a more adequate specification.

FRANK A. MCINNES. I should like to ask one question. I understood your test was designed so that the stem would cripple before the valve itself was broken.

H. O. LACOUNT. That is true. The part of the specifications just referred to shows what the stem must do. It must withstand the twisting moment as assembled in the valve. But going beyond that, the specifications do require that the stem shall finally cripple before other essential parts of the valve shall break. In other words, although the stem is just as strong as shown by the twisting moment, it yet is the weaker member in the valve construction as a whole, so that we will not run a chance of breaking the bonnet or putting the valve in some essential feature out of commission in advance of the stem crippling. The stem is more easily replaced than these other parts.

ROBERT SPURR WESTON.* Have there been great improvements in the bronzes, and is the bronze now used modern?

H. O. LACOUNT. I do not think I can answer that question in detail. I have no doubt that there has been some improvement.

I might have said that in putting this twisting-moment article in the specification, we were not feeling the necessity of going quite so high in the tensile requirements as the American Water Works Association. They require 45 000 lb. per sq. in. We have continued the original 32 000 lb.

It is generally difficult to obtain the field samples of broken stems for examination as they are usually thrown away but those we have seen showed fractures that certainly indicated either inferior metal or a real flaw. The twisting moment has brought to light quite a little of that inferior construction, and I think the valves as a whole have given satisfactory service in the field.

THE CHAIRMAN. Had there been many difficulties that you knew of prior to the adoption of your specifications, where the valve actually failed as the result of the stem nut giving way?

H. O. LACOUNT. We had so many failures that we were really disturbed about it. Our inspectors would report valves that were improperly closed. Among these would be indicator-post valves with the targets reading open, but no water passing from the riser through the valve in the test. Examination would show that the stem had been twisted off, or the lug on the stem nut had been broken, and the disc had dropped back into closed position and not a soul in the plant knew it. The trouble has been largely with the inside-screw type of valve. The outside-screw-and-yoke type we have not had as much trouble with. I think it may be said also that the valve which is out in the open does not get quite as hard usage as the valve which is buried, which is out of sight, where you get a wrench and go to it for all you are worth. We have had just a few reports from the so-called

*Consulting Engineer, Boston, Mass.

Underwriter valve where the stem or stem nut has been broken, but I think in those cases it was due to more than hard usage, to abuse, therefore. Of course you can break anything if you go at it hard enough. Since, in general, these specifications have been in effect only for new work, there are many of the older type of valves still in service. I just roughly counted the cases that have been sent in, with more or less detail of the accident and where we had five or six of these so-called Underwriter valves with the stems broken, there were about 50 of the commercial valves reported with the stems broken, in addition to many others for which trouble had been reported but in less detail.

JOHN T. ROBERTSON. May I ask whether all the commercial valves were made in accordance with the American Water Works specifications, or whether they were made in accordance with some private specifications of the manufacturer?

MR. LACOUNT. I can't answer that question definitely. I believe that the valves were of the commercial type, and I imagine that they reflected the American Water Works specifications perhaps more than any other specification, although I would not be surprised if the Water Departments here and there have what they call their own specifications, which perhaps follow the American Water Works specifications with some modifications. I do not think the valves to which I have referred were put in as particularly cheap valves, but believe they were representative water-works valves.

S. G. WILEY. I think it is safe to say that the leading commercial valve manufacturers follow the requirements of the American Water Works Association. While inspecting six different valves a short time ago I found the stem diameter of two of them was very much larger than on some of the others. On talking this observation over with one of the men representing the manufacturing company I was told that while the American Water Works specifications called for a certain size and strength of stem, there was nothing in the specifications to prohibit the stems being made larger or stronger if the manufacturer thought it necessary.

MR. LACOUNT. There is another matter of interest. I think one of the manufacturers quite frequently puts in so-called Underwriters' stems in the commercial line of valves. He thus tries to economize by using one line of stems instead of two. This practice does not constitute a detriment to the commercial line of valves.

Some time ago I talked about this matter with Mr. Gear of Holyoke, and he reported quite a little trouble with stems. He did not complain about the stem quite so much as he did about the fact that there was not room enough in the valve to accommodate a little rust and corrosion and stated that as a result an undue strain was placed on the stem when the valve was closed. That may be another item to be considered. I am inclined to think that if the stem had been a little stronger the gate would close without crippling the valve. It is suggestive, however, of the fact

that at some places there is trouble from the weakness of stems or stem construction.

THE CHAIRMAN. Has Waltham had any trouble?

GEORGE C. BREHM.* We have not had one breakage in ten years. However, we may expect failures. For instance, we had a fire in Waltham the other day, and I found a fireman standing at a hydrant "cussing" because he could not open it. I said, "Try and turn it the other way and see how it opens." It opened. Of course if valves are treated this way they will break.

THE CHAIRMAN. How about Peabody?

CHARLES J. CRUMP.† We never have any trouble.

W. E. MCDONALD. We have had some trouble with the twisting of hydrants. We have been overcoming that difficulty by the use of stainless steel. Since using this steel we have not had a failure. The use of stainless steel is working out very satisfactorily.

THE CHAIRMAN. To assist the officers of the Association I am going to ask for an informal vote as to whether this meeting thinks it worth while for us to follow up this subject to the extent of asking the Engineering Standards Committee to include the matter of valve stems in their discussions. Those who feel that it is will please rise. (*Nine members stand.*) Those who do not. (*Two members stand.*)

Will those who object please state their reasons?

W. C. HAWLEY.‡ If I want a watch I do not get up a set of specifications; I think over the names of the reputable watch manufacturers who are making good watches and I buy a watch with the guarantee of the concern that makes it. I am thus pretty sure to get a good watch.

In the matter of gate valves, there are plenty of concerns making good gate valves, and they will stand behind their product. I might specify the conditions under which the valves shall be operated, such as the working pressures, but I do not believe that we should attempt to tell the manufacturer what he must do, as long as he makes a valve which will operate under our conditions.

There is another angle to this subject. Perhaps I am fortunate in the plan that I have carried out for a great many years. I have chosen a valve with which I have had long and satisfactory experience, and I have given the manufacturer to understand that just as long as he furnishes valves that are satisfactory and stand up under our service conditions and just as long as he stands behind his goods and gives us reasonable prices, I am going to use his valves. In our system we have one make of valve and we do not have to carry but one set of repair parts. When I took my present position, we had some twenty different makes of meters, and we had twenty different sets of repair parts to carry, whereas we had one set of repair parts to carry

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†Superintendent of Water Works, Peabody, Mass.
‡Chief Engineer, Pennsylvania Water Co., Wilkinsburg, Pa.

for our valves. For these reasons I am not interested at all in the matter of specifications for gate valves.

M. J. RUTLEDGE.* In my experience I have found that the manufacturers, in contact with purchasing agents and water-works officials, develop a line of valves that is perhaps four or five years ahead of the engineering specifications. I, therefore, prefer to go to the manufacturer directly and to adhere to his specifications. To my mind, furthermore, standardization stops progress.

THE CHAIRMAN. I can see where a water company might prefer to take the stand just propounded, but I can also see the position of municipal superintendents. While some of the latter may be able to run their departments in the same way as a private water company, many of them may not be able to do so. Under these conditions, specifications would be very essential. Would not this situation make a difference?

W. C. HAWLEY. It certainly would. I have been in a position where I had to write the necessary specifications and use them. These specifications were general in character and proposals were asked from manufacturers of gate valves of a certain type. There was always, however, the difficulty of manufacturers who had pirated gate-valve designs on which patents had expired or who were manufacturing something "just as good." In buying a gate valve which will be built into a water main and over which a permanent pavement will be laid, the saving of a few dollars in first cost may be poor economy. Maintenance may in a few years cost more than the saving, to say nothing of the interruption of traffic and of service to water consumers. The best gate valve is none too good and the prices charged by the best manufacturers are not unreasonable.

*Town Manager, Woodstock, N. B.

THE GRANVILLE DAM AT WESTFIELD, MASS.

BY C. A. FARWELL.*

[*Read February 13, 1929.*]

Westfield is situated near the southerly boundary of Massachusetts, about ten miles west of the Connecticut River, in the County of Hampden, and near the larger city of Springfield. The more thickly populated section of the city is in the broad valley lying between the ridge which bounds the westerly side of the Connecticut Valley in this region, and the Berkshires. The Westfield Great River, which the Boston and Albany Railroad follows in climbing the easterly side of the Berkshires and which figures prominently in the headlines of the press during times of flood, flows through the city. Westfield Little River, which joins the Great River at Westfield, furnishes, in its upper portion, the "Little River Supply" of the city of Springfield where further development is under way at the present time. The Granville supply of the city of Westfield, which is being further developed by the Granville Dam, is also on the watershed of the Little River.

Population and Water Consumption. Westfield became a city in 1920 and had at that time a population of 18 600. The State Census of 1925 gave it a population of 19 342. The growth had been more or less regular from 1880 to 1915 but during the five-year period 1915–1920 there was practically no change, due, presumably, to the World War and its effect on local industry. The fact that Westfield had been the center of the whip industry in the United States may also have had its effect as, since 1915, most of our vehicles are no longer equipped with four-legged motors and whip sockets. There are, however, many other industries in Westfield, and there seems to be no reason to believe that the city will not grow at a normal rate in the future. The estimated population in 1927 was 20 647. From 1911 to 1926, the water used per capita per day averaged about 110 gal. The per capita consumption has been held down since 1926 to around 90 gal. per day. Only a small proportion of the services are metered — 6.7 per cent. in 1927 — but about 30 per cent. of the total water consumed passes through metered services. In estimates of future water requirements it has been assumed that the consumption of water per capita will increase gradually and will reach a figure of 140 gal. per day in 1955.

With a forecasted population of 37 100 and the per capita consumption of 140 g.p.d., the total water consumption for the year 1955 will be 1 900 million gal. or an average of 5.2 m.g.d. The year 1955 was used as the basis of design to insure that no further developments will be needed before indebtedness for those now under way is paid off. A demand of 6.7 m.g.d.,

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the fully developed capacity of the sources of supply, probably will not be reached within fifty years.

Present Water Supply. Westfield is fortunate in having a water supply that is delivered to the city at high pressure and by gravity. The supply is surface water of good quality comparing favorably with the supplies of Springfield, Holyoke, Chicopee and Metropolitan Boston. There are two sources, one about five miles northwesterly of the city in the town of Montgomery, the other about six miles southwesterly in the town of Granville. A plan is shown in Figure 1.

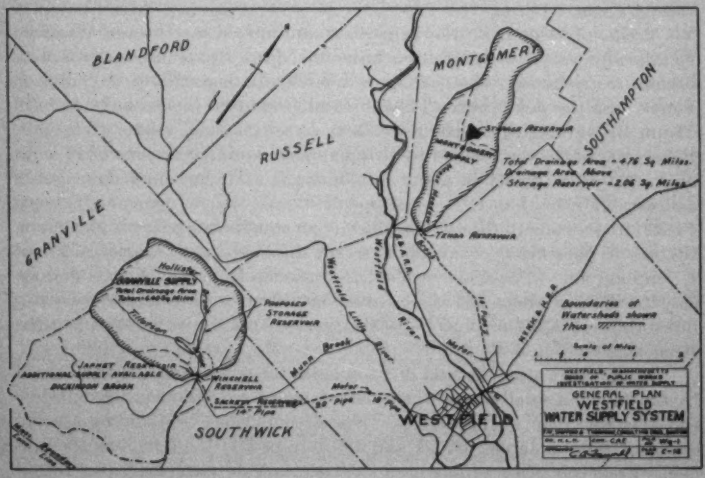

FIG. 1.— PLAN OF WESTFIELD WATER SUPPLIES.

The Montgomery Supply System, situated on Moose Meadow Brook, has a watershed area of 4.76 sq. miles, of which 2.06 sq. miles lie above the storage reservoir and 2.7 sq. miles between the storage reservoir and the diversion reservoir. The storage reservoir, formed by an earth dam across the brook valley, has an area of 38 acres and a storage capacity, as operated at present, of about 125 million gal. The spillway is at elevation 865, or about 775 ft. above the city. After leaving this reservoir the water flows along the bed of the brook for a distance of 2½ miles to the point of diversion — Tekoa Reservoir, 480 ft. below the storage reservoir. At Tekoa Reservoir, a masonry dam and head works divert the water into a 14-in. supply main leading to the city, a distance of about 4½ miles. This diversion reservoir has an area of 1½ acres and a storage capacity of about 4 million gal. Its spillway elevation is 386.2, being about 290 ft. above the street

level in the business section of the city. This supply system was built in 1874, and is said to have cost about $250 000.

The Montgomery Storage Reservoir was not stripped of loam at the time of construction, and as a result the water drawn from it contains considerable amounts of microscopic growths which at times produce a very noticeable color, taste and odor. These objectionable qualities are, however, practically entirely removed by the aëration the water receives in passing over the steep, rocky bed of the brook between the storage and diversion reservoirs.

During the past five years this system has supplied water at an average rate of about 1.15 m.g.d.

Under Acts of 1895, the Legislature authorized the town of Westfield to take any waters within the watershed of Munn Brook in Granville, with certain restrictions. About 1898, a diversion dam was built on Tillotson Brook near the point where Tillotson and Dickinson brooks unite to form Munn Brook and where the watershed area is 5.99 sq. miles. Winchells Reservoir, which was formed by this diversion dam, has an area of 1¾ acres and a storage capacity of about 3 million gal. The masonry dam has its spillway at elevation 410. A supply main, 14 in. in diameter, extends 14 200 ft. to Sacketts Reservoir which is an equalizing reservoir located on the line to Westfield. Sacketts Reservoir has a storage capacity of about 1.7 million gal. The water surface in Sacketts Reservoir when full is at elevation 374 or about 285 ft. above the city streets. From here the water flows through a 20-in. pipe 7 750 ft., and an 18-in. pipe 9 100 ft., to the street-distribution system.

A few years after the first development, this system was supplemented by the flow of a small brook which naturally entered Tillotson Brook below the diversion dam. The diversion was accomplished by building a small concrete dam on Japhet Brook and a 6-in. pipe line 1 300 ft. long to Winchells Reservoir, thus adding about 0.4 sq. mile of watershed area to the supply.

The Granville System with no storage, has a dry-weather capacity which may average for a month as low as 0.8 m.g.d. and which is, for shorter periods of time, only a fraction of this amount. Prior to the present undertaking, the Granville System had cost the city about $160 000.

Need for Additional Supply. . The total amount of water available at present from both the Montgomery System and the Granville System may be as low as 2 m.g.d. during a dry-weather period.

The actual consumption for the summer months of the years 1920 to 1926, inclusive, averaged well above 2 m.g.d., reaching a maximum monthly average of almost 2.7 m.g.d. in August, 1925. In fact, in only one month of this period — July, 1921, when the rainfall was 5.17 in. — did it average less than 2 m.g.d. From the fire-protection standpoint the situation was gradually getting more serious and the Underwriters had advised that additional supply should be provided. In the last few years a rigorous

campaign to economize in the use of water has reduced the consumption somewhat; but even though this economy is permanent, the demand will increase with the growth in population and the gradual increase per capita in use.

The use of meters upon a greater proportion of the services was considered but was not looked upon with favor. It is quite probable, however, that gradually the metering will be extended.

Development of Storage on Tillotson Brook. This branch of Munn Brook had already been partially developed as described and was without question the proper source to develop for additional supply. A weir was installed for measuring the flow of the brook a short distance above the small diversion dam and at a point which was later selected as the site of the storage dam. Records of flow are available for about ten years, 1908–1918. (See Table 1.) This period included one or more extremely dry years and has been considered the best indication of probable future flow. With sufficient storage, Tillotson Brook can be depended upon to furnish a continuous flow of about 5.5 m.g.d., as against a variable flow which at times may be as low as 0.75 m.g.d. without storage. The city's requirements from this source will not reach the figure of 5.5 m.g.d. for perhaps fifty years, and it was decided to provide storage sufficient to develop a continuous flow of 4 m.g.d. from this source, making, together with the Montgomery Supply, the total forecasted requirement for 1955.

To provide minimum flows of 4 and 5.5 m.g.d. from Tillotson Brook requires storage to the extent of 500 and 1 090 million gal. respectively. These figures are based on the flows from 1908 to 1918, inclusive, and are fixed by the dry period of 1911 and 1912.

TABLE 1. — YIELD OF TILLOTSON BROOK 1908–1918.
At Site of Proposed Storage Reservoir.
Average by Months in Million Gal. per Day.

	1908.	1909.	1910.	1911.	1912.	1913.	1914.	1915.	1916.	1917.	1918.
January	7.60	4.50	8.89	2.24	5.43	8.78	4.55	9.67	9.97	5.48	5.41
February	9.40	9.54	8.12	2.31	5.02	6.31	6.48	12.67	8.79	4.56	8.23
March	16.17	14.59	18.78	9.00	16.26	12.29	14.28	5.59	11.62	16.38	17.49
April	9.70	17.38	11.25	10.00	14.14	11.47	17.19	7.74	19.42	18.19	13.78
May	11.76	9.50	6.27	2.68	11.04	6.75	11.49	4.91	9.55	8.28	6.20
June	4.14	3.90	6.12	3.37	2.91	3.11	3.06	2.67	11.74	13.32	2.94
July	2.64	1.31	1.19	0.86	2.20	0.89	2.04	4.50	5.19	3.71	1.52
August	2.75	2.86	1.07	2.08	1.44	0.82	0.98	7.85	1.85	3.42	...
September	1.10	1.47	1.26	3.05	1.99	1.69	0.65	2.11	4.01	2.88	...
October	1.71	1.28	0.86	10.54	7.01	9.33	1.03	2.51	1.85	4.93	...
November	1.43	1.46	3.15	9.93	3.73	6.54	2.46	3.94	2.40	8.57	...
December	2.33	2.80	3.85	8.44	5.34	6.46	1.86	5.26	4.70	3.81	...

Gagings were made approximately at site of proposed dam, below Hollister Brook.

Site of Dam and Reservoir. At a point about 2 000 ft. upstream from the existing diversion dam on Tillotson Brook the site for the storage dam

was selected. About 1915, when it became apparent to the Westfield Water Department that additional development was necessary, studies of possible sites were made, and the present site was decided upon as being the most suitable. The late Professor Crosby, the geologist, who studied the problem thoroughly at that time, and others who were called in, agreed to the decision. The author understands that even before Westfield acquired rights in this brook, Holyoke investigated the site as a possible source of supply. In our investigations we were fortunate in having the services of Dr. Charles Terzaghi, Associate Professor of Foundation Engineering at the

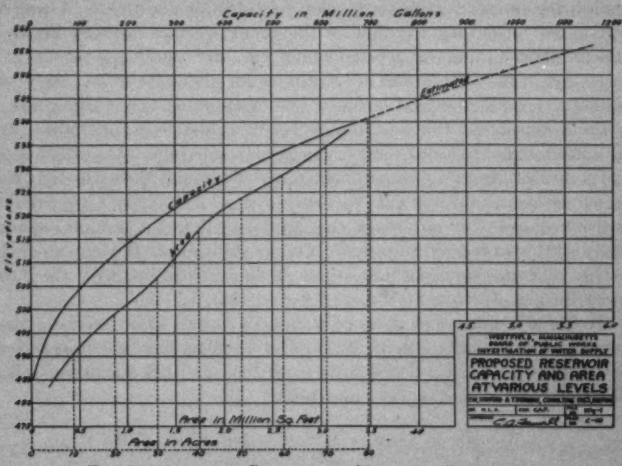

FIG. 2.— AREA AND CAPACITY OF GRANVILLE RESERVOIR.

Massachusetts Institute of Technology, on problems of geology and soil mechanics. Our studies verified the conclusions of earlier investigators. Above the site selected for the dam, the brook valley includes two brooks and has a wide floor with reasonably flat slopes, making a good reservoir site. Below the site the valley is narrow, steep-sided and falls rapidly; a dam farther downstream would need to be much higher to give the same storage. Curves showing the area and capacity of the proposed reservoir for different water levels are shown on Figure 2. It will be noted that a volume of 500 million gal. is included between elevations 538 and 507, this being the storage required for the draft of 4 m.g.d. forecasted for 1955, and that a volume of 1 090 million gal. is included between elevations 556 and 500, this being the storage required for a complete development of the watershed. The small amount of water stored below elevation 507 or 500 is not taken into account as it is very small and would be less suitable for use.

The geology of the site of the reservoir and dam is very complicated and

has been the subject of careful study. Numerous borings were made and test pits dug over the site of the dam and reservoir. Rock, a mica schist or gneiss, was found in the bed of the valley about 20 ft. below the surface. On the left or northerly side the rock extends approximately level beneath the slope, while on the right side it was exposed in a somewhat weathered condition in the lower portion of the slope, but after reaching an elevation of about 30 ft. above the valley floor it sloped gradually down to the southwest at increasing depths. Above the rock the material is glacial deposit varying greatly in different portions of the site. In the floor of the valley is an accumulation of boulders buried in a delta deposit. The right slope contains varying layers of fine sand, silty sand, and sand and gravel which, however, proved upon careful study to be of such character and so situated as to make it possible to secure a cut-off on this side that would insure against dangerous or large amounts of leakage. The left slope was quite different in formation. From the foot of the slope up to and slightly above the top of the proposed dam, the formation consisted of a soft reddish uniform sand, overlaid on the surface, however, by a hardpan blanket. Above the elevation of the top of the dam the hardpan was found to considerable depth. It was in this hardpan above the top of the dam that the borrow pit was later located. The explorations by means of test pits and borings were made before the location of the dam was accurately fixed and before designs were prepared. Another interesting geological feature and one of importance was the existence of an esker or porous formation of glacial gravel which crosses the reservoir site near its upper end and about 1 200 ft. from the dam. This formation leads from the reservoir off to the South and may result in the loss of some water from the reservoir. It is not expected that the amount of water lost through such leakage will be large but should the leakage prove serious, the reservoir wall at the esker can be blanketed.

It is not expected that there will be any leakage around the left wing of the dam. Around the right wing it is estimated that there may possibly be as much as 200 000 gal. per day but this is considered to be an outside figure. The amount of leakage that may be expected from the esker area is difficult to estimate and can be controlled as mentioned, if necessary. The seepage through the dam itself, taking into account the material that is being used in its construction, should not exceed 25 000 gal. per day. Careful studies have been made by Dr. Terzaghi of both the underlying material and that in the vicinity of the site of the dam and of that proposed for the embankment. Numerous tests were made on soil samples from borings and test pits to determine their permeability as a criterion of the water-tightness of the soil either in its natural condition or in embankment, and a standard of permeability was established for use in the field laboratory as a measure of suitability for material to be used in the embankment of the dam. A simple apparatus* was devised to permit quick field laboratory-determinations of the coefficient of permeability to be made.

*Described by E. B. Myott in the succeeding paper of this JOURNAL.

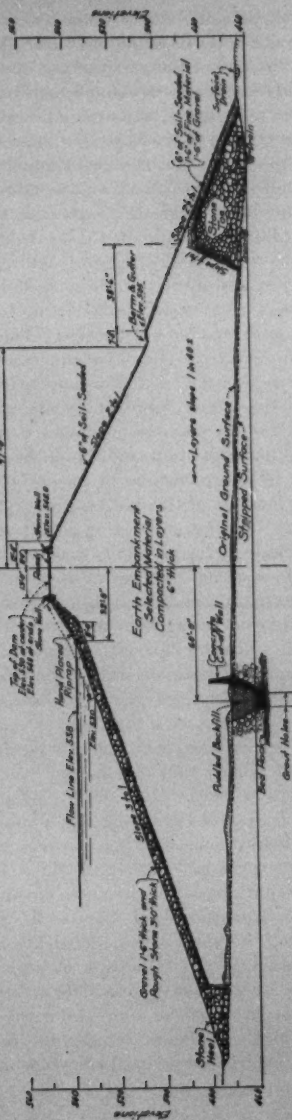

FIG. 3.— CROSS SECTION OF GRANVILLE DAM.

Constructions Provided. The new construction now under way provides a reservoir with a storage capacity sufficient to furnish 4 m.g.d. of water, the forecasted requirement from this source of supply in 1955. Provisions are made, however, for raising the dam and the reservoir level in the future an additional height of 18 ft., thereby providing for sufficient storage to completely develop the watershed.

The dam itself is of earth with a maximum height at the center of about 80 ft., a 3 to 1 slope on the upstream side, and 2 to 1 and 2½ to 1 slopes on the downstream side. The downstream toe is provided with a stone embankment and the upstream side with a stone facing. A cross section is shown in Figure 3. It will be noted that the upper of the two stonewalls built on the crest of the dam will in reality be a continuation of the stone facing and will act to increase the nominal freeboard of 10 ft. The earth available for the embankment was such that a core wall was not needed, the earth itself being sufficiently impermeable. The original plans called for a rather simple cut-off or key across the joint between the embankment and the underlying material to prevent seepage along this surface. However, construction operations disclosed the possibility of an undesirable amount of seepage through the material overlying the rock and also showed that the rock was traversed by seams to an extent not previously anticipated. In consequence, the cut-off now provided across the floor of the valley extends down from the surface to and into the rock, by means of grouting, and upward from the natural surface about 10 ft., with lesser construction on the slopes.

Beneath the stone toe at the downstream side of the dam, drains are provided to collect seepage water and to carry it to a central point where it can be observed and measured by means of a weir. All natural surface material and, in some instances, a layer of porous material were removed from the site of the embankment. Material for the embankment is in general being brought from the borrow pit located in the hardpan deposit on the left slope. This location was chosen as a result of the information disclosed by the test pits and to enable the material to be brought by downhill haul to the dam.

The outlet conduit is located in the rock near the bottom of the right slope. It consists of a 42-in. reinforced-concrete pipe and a 24-in. cast-iron pipe embedded in concrete, the two pipes being combined into one concrete structure. Particular care was taken to insure that there be no leakage along the outside of the structure, and concrete collars were provided on all sides at short intervals.

The 42-in. pipe serves to carry the brook flow during construction and later, during operation, to drain the reservoir or to blow off the sediment in the bottom. The 24-in. pipe is available during construction to assist in carrying floods or to pass water to the supply main if found desirable, and later, during operation, it will be used as the service outlet-pipe carrying water from the upper gatehouse to the supply main. In their use as

brook conduits during construction these two pipes together have a capacity, with a few feet of head at the upper end, of about 300 c.f.s., or 50 c.f.s. per sq. mile of watershed, which will care for anything but an extreme flood. The profile was so fixed that it will, under conditions of large flow, be below the hydraulic grade line, and its outlet is also kept well down in the bed of the brook in order that as much as possible of the velocity head will be regained. The entrance structure is located practically in the bed of one of the two brooks, the other brook being brought to it through a diversion ditch.

The flow through the outlet pipes is controlled at the upper gatehouse. Water may be taken into the gatehouse from the reservoir at one or more of four elevations, permitting it to be drawn from near the surface or from levels below the surface as desired. On the outside of the gatehouse tower, provision is made for coarse-screen racks through which the water must pass to reach the inlet pipes. Stop planks may also be placed at this location, in emergency, for repairs to inlet valves. From the chamber into which these inlet pipes lead, the water passes through screens into a second chamber, from which it is drawn off through either a 24-in. or a 12-in. valve to the 24-in. outlet pipe. This is the point at which the draft from the reservoir will be controlled, and the valves may be arranged to permit either automatic or distant control if such should appear to be desirable in operation. The valves shutting off the 24-in. outlet pipe above the gatehouse and those on the 42-in. blow-off pipe are also located in this gatehouse, both being in duplicate. All valve stems are brought up to the floor of the gatehouse where they are operated by valve stands, geared as necessary.

A hand-operated, overhead bridge crane is provided for handling all screen racks, valves, etc. The screen guides are of brass to insure ease of handling of screens. It is planned to use screens only in the bottom 10 ft. of the guides, the screen rack being designed to close the opening, except through the screen. If, however, this scheme should prove in operation not to be a success, racks can be used for the entire height up to water level as is customarily done. The floor of the gatehouse is at two levels, there being a low section around the screen guides where the screens may be washed. For safety, all long ladders are enclosed.

The gatehouse floor is reached from the crest of the dam by means of a foot bridge with trusses of minimum weight to permit of easy raising to a higher level when the dam and gatehouse are raised to the height required for final development.

On the downstream side of the dam, the 24-in. outlet pipe leads to a lower gatehouse where the water may pass, under proper control, either to the pipe line leading to Winchells Reservoir and a connection with the main to the city, or to a discharge into the brook bed. Provision is also made for connection to an aërator if one is found desirable in the future. A Venturi or other meter may also be installed here, if required.

The spillway is located at the southerly end of the dam on the right

bank. It is set well into the natural hill, and substantial cut-offs are provided to prevent seepage. The crest length of 60 ft. gives the spillway weir a capacity, with 4 ft. flowing over the crest, of about 2 000 c.f.s. or 330 c.f.s. per sq. mile of watershed. The above depth on the crest would still leave a freeboard of 6 ft., and this could be reduced without danger, but with the large reservoir to absorb the peak flow of a flood, the spillway has a large factor of hydraulic safety. The crest of the spillway was designed as a base for a future Ogee dam which will be built to a further height of 18 ft. when the dam is raised. The spillway is crossed by a bridge with a 12-ft. clear roadway carried on half-through trusses, these being made as light as possible to permit of raising the bridge to the ultimate height. From the spillway crest the channel follows a grade contour to the brook bed at a point well below the downstream side of the dam. The 60-ft. width at the crest decreases in about 175 ft. to a width of 10 ft. on the bottom. The typical cross section, having a 10-ft. bottom width and $\frac{3}{4}$ to 1 side slopes, is used throughout the remainder of the channel, the depths varying from 8 ft. to 5 ft. depending on the slope. At the lower end is a structure which will serve to break up the flow during heavy discharge. Numerous weep holes are provided in the sides and bottom of this channel to prevent lifting of the lining by hydrostatic pressure, and concrete collars are provided at short intervals to insure that water will not find a channel behind the lining.

The flow line of the reservoir is to be "stripped" of loam through a vertical distance of 12 ft. and is to be "grubbed" of all vegetable matter to a further depth of 18 ft., measured vertically. In addition certain swampy areas are to be "blanketed" with sand or gravel.

It has been necessary to use water from this source during construction and great care has been taken to keep it safe from contamination. Not only are the contractor's forces under strict sanitary regulation, but the water is chlorinated.

Roads were laid out to connect with each end of the dam, the road on the southerly end being part of a circuit which will enable visitors to obtain a good view of the dam and reservoir. The road across the dam and that along the northerly side will, for the present at least, be service roads only.

Contract and Specifications. In preparing the contract the work was divided into a number of items upon most of which unit-price bids were received. In addition, another item, "establishment and dismantling expense," was included, to enable a bidder to cover costs of a general nature under this overhead item allowing the unit prices to closely represent the costs on the units. With this provision, a variation in estimated quantities in the particular units of work should not result in unfairness to either party.

Separate items were provided for excavation and embankment to permit of better control over the material used in the dam. This means, of course, that material used is paid for under both of two items. The furnishing of pipe and valves was not included in the construction contract; they were purchased by the city under other contracts.

Cost. The total expenditure for the work is estimated at $630 000, of which about $525 000 covers the contract work described, including some additional work not originally included in the contract.

Organization. The work is being done for the city of Westfield, of which the Honorable Louis L. Keefe is Mayor, and is under the direct charge of the Board of Public Works, consisting of Messrs. John O. McKean, Chairman, John L. Barry and Edward E. Hinsman; Mr. Oren E. Parks, a member of this Association, who is Superintendent and Engineer for the Board, had an active part in the design and is in general charge of construction for the Board; the C. & R. Construction Company of Boston are the contractors on the work; the author's firm, Fay, Spofford and Thorndike, is serving as Consulting Engineers, Mr. E. B. Myott being Resident Engineer at the site during construction; Dr. Charles Terzaghi has served as consultant on geology and soil mechanics.

CONSTRUCTION OF THE GRANVILLE DAM AT WESTFIELD, MASS.

BY E. B. MYOTT.*

[*Read February 15, 1929.*]

Magnitude and Present Status of Project. The construction of the Granville Dam which is now under way involves a total expenditure, exclusive of the purchase of land, of $630 000. The project comprises the building of an earth dam and appurtenant structures, the preparation of the reservoir site for the storage of water, and the building of a short supply-main. Most of the work is being done under contract.

Construction bids were received on September 12, 1927, and the contract was awarded to the low bidder October 28. Fifteen contractors submitted bids, ranging from $430 000 to $800 000, the lowest being 26 per cent. below the average.

Construction work began in the late fall of 1927 and continued to February, 1928. Only a small amount of work was accomplished during this period, namely, the construction of a contractor's camp, the laying of about 600 ft. of 20-in. pipe, and the erection of a cableway. Work began in earnest in April, 1928, and was carried on as fast as practicable until December, when all operations were suspended on account of unfavorable weather. During this time work costing $350 000 was completed. The indications are that the project will be completed well within the specified time which ends October 31, 1929.

Order of Operations. The order of operations decided upon required the completion of all concrete work and a small amount of the embankment during 1928. Although it was necessary, due to the character of the foundation material, to make much more rock excavation and place more concrete than was originally planned, all the concrete work was practically finished, and about one-quarter of the embankment was placed.

Work was first started at the dam on the outlet conduit. Simultaneous construction of the cut-off wall, preparation of the dam site for the embankment and construction of the upper gatehouse then followed. During the latter part of the season, excavation for the road on the south bank of the dam and construction of the spillway and spillway channel and the lower gatehouse were nearly completed; the borrow pit was opened, the earth embankment was placed to an average height of 16 ft.; a considerable amount of work was done in preparing the reservoir site for the storage of water; and the 20-in. supply-main was finished.

Outlet Conduit. The outlet conduit consists of a reinforced concrete

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pipe 42-in. in diameter, and a 24-in. cast-iron pipe embedded in concrete. The conduit is about 740 ft. long and is built in rock trench varying in depth from 3 ft. (outside the dam) to 25 ft., averaging about 15 ft. (Fig. 1.) The flow of the brooks is diverted into the conduit through a special inlet structure shown in Figure 2 and discharges into the old brook channel at a considerable distance below the dam. The conduit passes through the upper gatehouse where gates will control the flow of water in it.

FIG. 1.— OUTLET CONDUIT.

Construction of the conduit required close engineering supervision as it was necessary to establish the profile while trench excavation progressed. The rock encountered along the conduit was granitic gneiss, with many irregular seams, and badly disintegrated in many places. This condition not only increased the cost of excavation to the contractor, as a large amount of rock had to be excavated without blasting, but made it necessary to remove much more rock and use much more concrete than anticipated. Only light charges for blasting were permitted by the specifications, which were strictly enforced. The depth of all holes averaged only 2.6 ft., and the amount of dynamite used was kept to a minimum.

The conduit was built in sections averaging 40 ft. in length. Concrete was first poured to the springing line of the 42-in. conduit. The second pour was then made to a height of 9 in. above the crown, and following this the cast-iron pipe was laid and covered with concrete. Semicircular wooden forms were used for the 42-in. conduit. They were built in sections 10 ft.

long, with 2-in. x 8-in. ribs spaced 2 ft. on centers, lagged with $\frac{7}{8}$-in. x $1\frac{1}{8}$-in. stock. Concrete collars were built about every 30 ft. at the bottom and sides of the trench and to the top of the banks of the trench, to prevent any seepage along the conduit. (Fig. 3.)

A 400-ft. cableway was used for removing the excavated material and handling the cast-iron pipe and other construction materials over the deepest part of the trench. A Byers crane served this purpose on the re-

FIG. 2.— INLET STRUCTURE OF OUTLET CONDUIT.

mainder of the trench. Most of the concrete was poured from portable mixers located along the bank. Construction was completed in four months at an average cost of $67 per lin. ft.

Cut-off Wall. It was thought, previous to construction, that only a simple cut-off wall would be required. The contract drawings called for a concrete wall built across the floor of the valley in as narrow a trench as possible, resting on bed rock and extending above the surface of the ground about 5 ft. As bed rock is very deep on the banks of the valley, it was planned to use timber sheet-piling together with a simple wall or an earth cut-off at these points. Since it was found, however, that timber piling could not be driven, it was decided to build the cut-off of concrete for the entire length of the dam.

Excavation for the cut-off wall was first started on the south side of the valley floor. As soon as bed rock was encountered, the rock was tested for water-tightness. One-inch holes 10 ft. deep took water at the rate of

about 5 g.p.m. under a 20-ft. head on the bottom of the hole. This fact, together with knowledge obtained during construction of the outlet conduit regarding the character of the rock, caused a change in the design of the wall.

The material overlying bed rock, across the valley floor, is made up of silt, sand, gravel and large boulders and is very compact; on the north bank there is a layer of hardpan overlying soft, reddish, fairly uniform sand; on

FIG. 3.— CONCRETE COLLARS ON OUTLET CONDUIT.

the south bank the material consists of silt, grey granitic sand and fine gravel; at the southerly end of the dam there is a considerable amount of rock-flour silt.

The trench across the valley floor was opened to an average width of 8 ft. at the bottom and 20 ft. at the top, but on the banks of the valley the trench was kept to a minimum width of about 3 ft. (Fig. 4.) A seal coat of concrete was poured over the bed rock, and the rock was grouted. The cut-off wall was built on the downstream side of the trench in sections 40 ft. long; below the ground surface the cut-off was built in steps. Walls were built across the trench at the end of each section, making it possible to place the puddle backfill, which consisted of rock-flour silt and topsoil, in sections. The cut-off wall was built to an average height of 10 ft. above the ground. Particular care was taken in grouting the foundation and in constructing the cut-off at its intersection with the outlet conduit. On the banks of the valley the trench depth varied from 12 ft. at the foot of the bank to 9 ft. at the top, and the height of the wall above the ground surface varied from

10 ft. at the foot of the bank to 5 ft. at the top. Passageways were left in the wall for hauling the embankment material to the dam.

The entire length of exposed bed rock was grouted. Use of a set of 1-in. steel drills gave a diameter of hole at the top of about 3 in. The holes were spaced 15 ft. center to center and staggered in two lines about 7½ ft. apart. In all, 60 holes were drilled, 52 of which accepted grout. At the beginning of the grouting operation, the grout was mixed in the proportion of one part cement and seven parts water. As the hole tightened, the mix

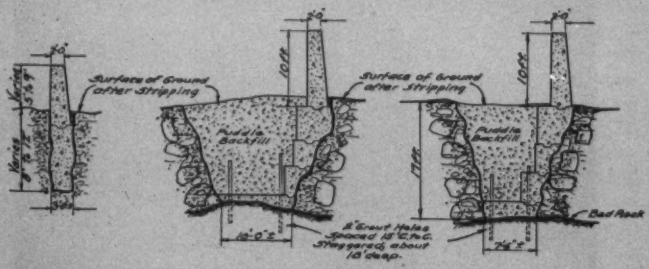
Surface of Ground after Stripping
Puddle Backfill
Surface of Ground after Stripping
Puddle Backfill
Bed Rock
4" Grout Holes Spaced 15' C. to C. Staggered, about 15' deep.
ON BANKS OF VALLEY
ACROSS FLOOR OF VALLEY

FIG. 4.— CUT-OFF WALL.

was gradually increased to one part cement and three parts water. The grout was first applied under a pressure of 15 lb. per sq. in. and this was gradually increased to 80 lb. The minimum amount of cement used per hole was ½ bag, the maximum 75 bags and the average 10¼ bags. Only a small amount of grout was lost. The total cost of grouting was $2 475 or about $4.25 per lin. ft. of hole. At the present time, the cut-off wall is practically completed. Four months were required to build the wall and the average cost per lin. ft. was $92.

Upper Gatehouse. The upper gatehouse is a reinforced concrete structure 22 ft. x 30 ft. in plan and approximately 85 ft. high, provided with a one-story brick superstructure. The gatehouse contains a dry well and a wet well. The wet well does not extend to the very bottom of the structure; a large section of mass concrete about 28 ft. high lies beneath. The reinforced walls for a height of 28 ft. above the foundation are 4 ft. thick; above this they are 3 ft. thick.

The gatehouse was built on rock foundation of the same character as that encountered in the outlet-conduit trench. It was necessary to excavate more rock and to place more concrete than originally estimated. The concrete in this structure was placed in pours as large as practicable, averaging about 100 cu. yd. per pour. The concrete for the lower portion of the structure was poured by gravity from a mixer located on the south bank; that in the middle portion was handled from the mixer by a crane;

a steam-hoist elevator was used to handle the concrete in the upper portion. The forms consisted of 2-in. x 6-in. spruce planed on one side with 3-in. x 4-in. studs, 2 ft. center to center and 4-in. x 6-in. ribbons, 3 ft. center to center. The forms were double wired on 3-ft. centers both ways. (Fig. 5.)

The progress of the work depended upon the construction of the forms, and as only a limited number of men were able to work on the forms at one

FIG. 5.— GATEHOUSE.

time the building progressed slowly. The total cost of this structure will average about 95c. per cu. ft. of gross volume.

MAIN DAM.*

The dam is an earth embankment built by the layer-fill method, with a stone toe and heel, and stone facing on the upstream slope. There will be used 240 000 cu. yd. of earth, 30 000 cu. yd. of stone, 8 000 cu. yd. of gravel and 2 000 cu. yd. of soil dressing. The stone embankments and facing have been designed so as to utilize all the stone excavated for appurtenant structures and all stone encountered in the borrow pit. The average cost per cu. yd. of the entire embankment will be about 85c.

Preparation of the Site. The first work in the construction of the main dam was the preparation of the site for the embankment. This included the removal from the site of all stumps, topsoil, loose boulders, and material

*See Figure 3 in the preceding paper of this JOURNAL: "The Granville Dam at Westfield, Mass."

which in its natural state was quite permeable. A large part of this work was done by a steam shovel. The final operation of removing the loose material left by the shovel was accomplished by hand labor. The work progressed slowly at first due to the difficulty in hauling the material away. A Linn tractor was of particular value in this connection and had no difficulty in hauling a full load over rough ground on a grade of 20 per cent. Many boulders were encountered in stripping the site. Their removal required a large amount of hand labor. All loose boulders were used in building the stone toe of the dam.

The Borrow Pit. While preparation of the dam site was under way, a road was excavated to the borrow pit, and a part of the pit was grubbed and stripped. The pit is located on the north bank of the valley about one-quarter mile from the center of the dam on a side hill with an average cross slope of about three to one. Preliminary investigations showed that the side hill was covered with a layer of hardpan approximately 10 to 15 ft. thick at its junction with the valley floor, and 60 to 80 ft. deep at the top of the hill. Underlying the hardpan a large volume of uniform reddish, micaceous sand, which is very permeable, and therefore unsuited to use in the dam embankment, was encountered.

The specifications permitted the contractor to open the borrow pit at any point he deemed desirable on the side hill, but not within about 300 ft. of the embankment. The contractor elected a spot approximately at elevation 550, about 10 ft. above the top of the dam, where the depth of the overlying hardpan would be great enough to permit economical excavation and handling of the material. The road from the pit to the floor of the valley at the dam site was built on an average grade of 10 per cent. As the height of the embankment increases, the alignment of the road will be changed somewhat, thus decreasing the grade of the road.

The borrow-pit excavation is being carried to the entire depth of good material by two Byers steam shovels, equipped with ¾-yd. buckets. These shovels are capable of excavating the bank to a height of about 25 ft. It was first planned to terrace the cuts with depths not greater than could be made by the shovels in one cut, but this idea had to be abandoned on account of the difficulty of handling the hauling equipment in the pit. When the height of the bank reaches about 25 ft., a cast-over cut is made from the top of the bank by a Byers gasoline shovel equipped with a 1-yd. bucket. This shovel casts the material over the bank where it is rehandled by the steam shovels before another cut into the bank is made. It is not necessary to use the gasoline shovel on this work all the time.

The hardpan is very difficult to excavate. Although the teeth of the buckets are made of manganese steel, it is necessary to change them frequently. Excavation of this hardpan could be made more economically by larger and more powerful shovels, but the size of the shovels in use is well suited to the general scheme of handling the material. The shovels are quite speedy and are capable of excavating the material as fast as it can

be properly placed in the embankment. If larger shovels were used, additional spreading, rolling and hauling equipment would be needed.

During the past season the steam shovels worked 104 shovel days, averaging about 600 cu. yd. daily per shovel. During the last month of work they averaged about 760 cu. yd. daily. These figures do not represent the maximum capacity of the shovels, because all the equipment for hauling the material to the dam did not arrive until the last few days of the season. It is expected that during the next season the shovels will average about 900 cu. yd. per working day.

Hauling of Material from the Borrow Pit to the Embankment. At the beginning, the contractor was undecided on the type of equipment to use for hauling the material from the pit to the dam. Numerous types were considered, such as belt conveyors and trucks, auto trucks, Linn tractors, and tractor trains. The contractor purchased a 7-ton Linn tractor for general use on the job, and as soon as the earth embankment was started this machine started hauling from the pit. A 60-h.p. Caterpillar tractor, manufactured by the Caterpillar Tractor Company, and two bottom-dump steel wagons, manufactured by the LaPlant-Choate Manufacturing Company, were purchased on trial. Three-ton Mack trucks equipped with dual-pneumatic tires on the rear wheels were hired. Time studies were then made to determine the comparative cost of hauling the material by these different machines. The results of these studies are shown in Table 1. The studies were made at the beginning of the work when hauling condi-

TABLE 1. — TIME STUDIES AND COST OF HAULING EARTH FROM BORROW PIT TO EMBANKMENT.

Equipment.	Number of Trips.	Average Time Per Trip.				Trips Per 9-Hr. Day.	Total Cost of Equipment Per 9-Hr. Day.	Cu. Yd. Hauled Per 9-Hr. Day.	Cost Per Cu. Yd.
		L'd'g.	Lost.	Haul'g.	Total.				
Tractor Train No. 1	30	5′ 34″	4′ 04″	8′ 28″	18′ 06″	...	...	...	...
Tractor Train No. 1	38	4′ 13″	0′ 34″	9′ 02″	13′ 49″	...	...	...	...
Tractor Train No. 2	18	5′ 13″	3′ 30″	9′ 47″	18′ 30″	...	...	...	...
Average for Trains	...	...	...	...	16′ 48″	32	$40	320	$0.125
Truck No. 3	52	1′ 37″	2′ 19″	6′ 47″	10′ 43″	...	...	...	...
Truck No. 4	52	1′ 44″	2′ 15″	6′ 57″	10′ 56″	...	...	...	...
Truck No. 4	49	1′ 23″	2′ 20″	7′ 05″	10′ 48″	...	...	...	...
Truck "A"	32	1′ 19″	1′ 50″	7′ 36″	10′ 45″	...	...	...	...
Truck "A"	47	1′ 25″	2′ 19″	7′ 48″	11′ 32″	...	...	...	...
Average for Trucks	...	...	...	...	10′ 57″	49	$27	122	$0.225
Linn Tractor	28	3′ 0″	0′ 34″	7′ 45″	11′ 19″	...	...	...	...
Linn Tractor	21	2′ 40″	0′ 55″	11′ 39″	15′ 14″	...	...	...	...
Average for Linn Tractor	...	...	...	...	13′ 17″	41	$36	184	$0.195

A Train consists of a 60-Horse Power Tractor and 26 Cu.-Yd. Dump Wagons.
Capacity of Truck = 3 Tons.
Capacity of Linn = 7 Tons.
Total Haul = ½ Mile.

tions at the embankment were poor and the pit was not large enough to permit of easy handling of the equipment. The table shows that the earth can be hauled cheapest by tractor trains, the cost per cu. yd. averaging about 12½ c., while the figures are 19½ c. for the Linn tractor and 22½ c. for the trucks.

FIG. 6.— CONSTRUCTION OF DAM EMBANKMENT.

These studies proved conclusively to the contractor that tractor trains should be used. Consequently, additional tractors and wagons were ordered at once. As hauling conditions at the embankment and in the borrow pit improved, it was found that the Caterpillar tractors could haul 3 wagons and the Linn tractor could haul one wagon in addition to its usual load. Additional wagons were therefore ordered. At the end of the season, the material was being hauled by 3 tractor trains of 3 wagons and by the Linn tractor and one wagon. It is expected that the remainder of the filling material will be hauled by this equipment only. The cost of hauling the material with trains of 3 wagons and with the Linn tractor and one wagon will of course be much less than that shown in the tabulation. The cost per cu. yd. for the trains will be about 9c. and for the Linn tractor and one wagon about 13c. These figures do not include the contractor's overhead and other general charges.

Work on the Embankment. The work on the embankment requires spreading and rolling the material, as well as the sorting from the earth of

roots over ½-in. in diameter and stones over 6 in. in greatest dimension. A 60-h.p. Caterpillar tractor equipped with a LaPlant-Choate bulldozer spreads the material. A 7-ton Buffalo-Springfield roller assists the other equipment in compacting the fill. A 1½-ton truck, a foreman and five men are employed to pick out the roots and stones, and direct the dumping of earth and the placing of stone and gravel. (Fig. 6.)

Amount of Compacting. Perhaps the most essential factor in earth embankments is the compactness of the mass. For the greater the compactness the greater the weight, and in most cases the greater the imperviousness.

The specifications for this work read as follows:

"the filling material shall be distributed in approximately horizontal layers not exceeding six inches in thickness when compacted . . . . The compacting shall be done by approved rollers having grooved or banded rolls and operated by mechanical power except that in small embankments and around the structures other means approved by the Engineer may be used. The heaviest wheels of the roller shall cause an average pressure of at least two hundred fifty pounds per linear inch width of roll. The roller shall pass over every portion of each layer as many times as may be necessary, but not *less* than six times in any case."

This statement conforms closely to the specifications commonly used for the building of layer-filled embankments. At the present time specifications for this kind of work do not commonly include an allowance for compacting done by equipment other than that assigned to do the rolling. Before motorized equipment became so common, the compacting of embankments was done by horse-drawn or steam-driven rollers and by the trampling of stock. In the early dams, the material was hauled to the embankment in most cases by horses and wagons; later by industrial trains. Today when motorized equipment is used, such as that employed at Westfield, consideration should be given to the amount of compacting which is accomplished by it. It is true that this type of compacting is not entirely uniform, but it can be quite carefully controlled.

The engineers on this project were pleased when the contractor decided to use tractors and the large wagons which have been described. Realizing that the compacting done by a 10-ton tractor working on the embankment must be beneficial, studies were made to determine the relative amount of compacting accomplished by the rolling, spreading and hauling equipment. The results of these studies are shown in Table 2.

The distance travelled and the area covered on the embankment during a 9-hr. day, while placing a layer 5½ in. thick after being compacted, together with the wheel loads of all equipment, were determined. With these data and the knowledge of the area of the embankment, the average number of trips made on the embankment per layer was computed for each machine. The product of the average number of trips and the average wheel load per lin. in. width of tread gives the relative amount of compacting.

The relative amount of compacting ascertained for the tractor treads should possibly be reduced to conform more closely to the amount of compacting given for wheel loads, due to the additional concentration of a wheel load. However, considering wheel loads only, the tabulation shows that the amount of compacting done by the wagons alone exceeds the minimum requirements of the specifications. When work is resumed in the spring, we hope to extend this study to determine the absolute amount of compacting done by all machines.

TABLE 2.— RELATIVE AMOUNT OF COMPACTING DONE BY ROLLING, SPREADING AND HAULING EQUIPMENT FOR ONE COMPACTED LAYER 5½ INCHES THICK.

Equipment.	No. of Machines.	Total Width of Treads (In.).	Distance Travelled on Embankment (Ft.).	Area Covered By Treads (Sq. Ft.).	Wheel and Tractor Loads (pounds). Total. Loaded.	Total. Empty.	Total. Average.	Per Lin. In. Width of Tread. Loaded.	Per Lin. In. Width of Tread. Empty.	Per Lin. In. Width of Tread. Average.	Average Number of Trips.*	Trip Pounds Per Lin. In. Width of Tread.
Roller	1											
Front Wheels		37	35 000	108 700	4 960	4 960	4 960	134	134	134	1.04	140
Rear Wheels		36	...	104 300	9 900	9 900	9 900	275	275	275	1.00	275
Caterpillar Tractor	3	32	18 150	145 500	20 000	20 000	20 000	625	625	625	1.39	870
Caterpillar Tractor with Bulldozer	1	32	70 000	187 000	27 140	27 140	27 140	850	850	850	1.79	1 525
Wagons	10											
Front Wheels		20	18 150	303 000	8 000	2 440	5 220	400	120	260	2.91	755
Rear Wheels		24	...	363 000	15 350	4 710	10 030	640	200	420	3.48	1 460
Linn Tractor	1	28	18 150	42 300	28 250	9 250	18 750	1 010	330	670	0.41	272
Total Relative Amount of Compacting												5 297

*The average number of trips is the ratio of the area covered by the treads to the area of embankment.

In the light of these studies, specifications for layer-filled embankments should be written to take into account the equipment to be used and should be complete enough to include an allowance for the compacting done by the hauling equipment. On this particular enterprise, much more compacting is being done than was specified; but if the contractor had decided to haul the material to the dam by industrial trains or by other means, which would not give the amount of compacting equal to that represented in the tabulation, additional rolling equipment would have been required.

Numerous test pits have been dug in the embankment. Below a depth of 1 ft. it is impossible for a laborer to dig out the material without the use of a pick. Laborers say that it is as difficult to dig as any natural material with which they have ever dealt. Unfortunately, we have not at the present time figures that we can safely rely upon as measuring the amount of shrinkage of this material. At the end of this undertaking, it is expected, however, that quite definite figures will be available.

Soil Tests. The various kinds of soil encountered are tested in the field office by a simple method worked out by Dr. Charles Terzaghi of the Massachusetts Institute of Technology. The method involves the use of a device* for measuring the capillary rise of water in soil. The device consists of a small water tank, an irrigator tank, and a set of glass tubes. In

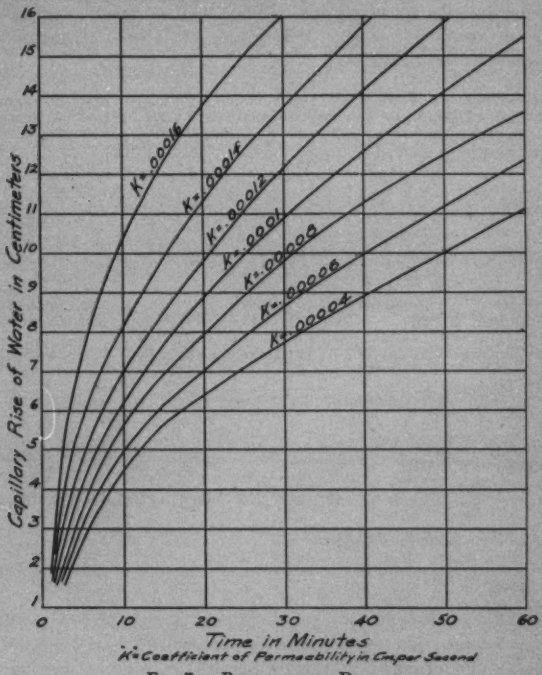

FIG. 7.— PERMEABILITY DIAGRAM.

making a test, the sample of earth is first dried, care being taken not to burn the material. The tubes are next filled with the dry material in layers that are compacted. The tubes are then placed in the tank, and water from the irrigator tank is fed to the tank containing the tubes. The dam at the end of the tank keeps the water at such a level that the bottom of the tubes is always submerged. The tubes are graduated or a scale is erected in back of them. Noting the time when the tubes are first placed in the water, readings are taken every five minutes as the water rises in the soil. These readings are then plotted on a diagram (Fig. 7) which gives the relation of the capillary rise of water in soil to the coefficient of permeability.

*See Figure 12 in the succeeding paper of this JOURNAL: "Soil Studies for the Granville Dam."

This diagram is based on the results of the numerous laboratory tests conducted previous to construction. The weight and volume of the samples tested are measured. Knowing the specific gravity of the material, the volume of voids and the voids-ratio of the material are then readily computed. The theory of the test is complicated, but its application is very simple. Taking a set of samples and testing and recording the results only requires the time of one man for about one-quarter of a day. The greatest

FIG. 8.— SPILLWAY.

value of the test lies in the fact that one can determine within an hour the suitability of the material.

Last season, 425 samples of material were tested in our field office. Of this number, 264 were of material being used in the dam embankment. The average coefficient of permeability of the 264 samples was .00007 cm. per sec. and the volume of voids 34 per cent. The line of saturation and the amount of leakage expected was computed on the basis of an average coefficient of permeability of .0001 cm. per sec. The material used is somewhat better than originally anticipated.

A system of pipe drains has been built underneath the stone toe, the main drain of which passes through a catch basin just below the dam. Ditches that will collect the surface drainage on the face of the dam will discharge into this catch basin. A weir has been built in the catch basin so that the amount of leakage through the dam can be measured. Weirs have also been constructed on each brook at the head of the reservoir. During the construction period, daily records are kept of the flow of water

over these weirs and also of the flow of water in the brooks after they join at the mouth of the outlet conduit. If records are kept of the amount of water drawn from the reservoir after construction and the amount of water entering the reservoir, together with the elevation of the water in the reservoir and the amount wasted, the approximate amount of leakage through the walls of the reservoir may be determined. Such a figure will be of particular value in checking the estimates made of the leakage by the geologists.

FIG. 9.— LOWER END OF SPILLWAY CHANNEL.

Miscellaneous Structures. The lower gatehouse, a small structure 16 ft. by 20 ft. in plan, consisting of a concrete basement and brick superstructure, houses gates that will control the flow of water in the supply-main. Provisions have been made in the piping layout to return the water to the brook or, if needed, to an aërator.

The supply-main extends from the lower gatehouse along the brook a distance of one-half mile to a connection with an existing main. It consists of a 20-in. cast-iron pipe laid in a trench with an average depth of 7½ ft., located on a steep side hill. The total cost of this pipe line per lin. ft. was about $11.50.

The spillway and spillway channel were built on the steep south bank of the valley. (Fig. 8.) At the upper end a large amount of rock-flour silt was encountered and the sides of the spillway were built as gravity retaining-walls. Sand, gravel and rock were encountered along most of the

distance. The slopes of the channel are paved. (Fig. 9.) Rock was encountered at the lower end of the spillway where the alignment is on a curve. The channel was built in sections 30 ft. long and on chords around the curves. Movable forms were used for the slope paving. At the lower end of the channel, difficulty was encountered in hauling the material to and from the channel as the grade here is 18 per cent. The only hauling vehicle which could operate on this grade was the Linn tractor.

TABLE 3. — TYPE, MAKE, SIZE AND COST OF EQUIPMENT USED.

Machine.	Makers.	No.	Rating.	Cost.	Remarks.
			Earth Excavating Machinery.		
Gas Shovel	Byers	1	1 cu. yd.	$15 500	With 50-ft. boom and drag line equipment.
Steam Shovel	Byers	2	¾ cu. yd.	24 000	
Bear Cat	Byers	1	...	8 500	With 30-ft. boom skimmer, ditcher and ½-cu. yd. Owen clamshell bucket.
Tractor	Caterpillar	1	"60"	6 450	
Tractor	Caterpillar	3	"60"	14 700	
Trailers	LaPlant-Choate	10	5 cu. yd.	12 000	Bottom dump.
Tractor	Linn	1	6 cu. yd.	8 200	Dump body.
Grader	Wehr	1	...	1 400	Fordson power.
Roller	Buffalo-Springfield	1	7 ton	3 500	4-Cylinder gas.
Total				$94 500	
			Concrete Machinery.		
Mixer	Rex	1	3 bags	$2 200	
Mixer	Lakewood	1	4 bags	3 100	
Mixer	Lakewood	1	2 bags	1 450	
Mixer	Rex	2	1 bag	1 950	
Sand Loader	Conant	1	...	300	
Total				$9 000	
			Miscellaneous Equipment.		
Lighting Plant	Koeler	1	0.9 k.w.	$500	Automatic generating.
Cableway	...	1	400 ft.	3 500	Steam boiler.
Compressor	Gardner-Denver	1	220 cu. ft.	3 300	
Compressor	Ingersoll-Rand	1	160 cu. ft.	2 650	
Stone Drills	Ingersoll-Rand	6	...	1 020	Air driven.
Stone Drills	Gardner-Denver	3	...	570	Air driven.
Truck	Indiana	1	1 t., 1 cu. yd.	1 800	Dump body.
Truck	Indiana	1	1½ t., 1½ cu. yd.	2 350	Dump body.
Truck	Indiana	1	3 t., 3 cu. yd.	3 500	Dump body.
Grout Mixer	...	1	...	375	
Pumps	Gould	2	4″x5″—21 g.p.m.	900	21 g.p.m. at 175 ft. head.
Total				20 465	
Grand Total				$123 715	

A road has been excavated from an existing road to the south end of the bridge. This road will cross the spillway and the dam. A pony truss bridge with a concrete deck has been built across the spillway. (Fig. 8.) The steel for the bridge was fabricated in Worcester and the trusses, 60 ft. in length, were transported via highways to the site. Only a small amount of trouble was encountered in guiding the truck and trailer around the sharp curves approaching the job.

A considerable amount of material excavated from the road and spillway channel was not suitable for use in the embankment. This material was employed for blanketing swampy places in the reservoir. In preparing the reservoir site for the storage of water, some work has also been done stripping the flow line. Most of the material on the banks of the reservoir is sand and gravel. A Byers gasoline shovel has been used in making a single cast-over cut along the flow line. This results in thorough stripping for a depth of about 10 ft. below the flow line and blanketing a similar depth below the cut. This method of treating the flow line eliminates a great amount of hauling of the stripped material and gives the shores of the reservoir a better appearance.

General. The contractor, the C. & R. Construction Company of Boston, has coöperated fully at all times with the engineers on this project. The job has been exceptionally well equipped at all stages of the work. As shown in Table 3, the total cost of all equipment used amounts to $124 000.

Prior to beginning the construction work which has been described, the city did a considerable amount of work in preparing for a reservoir in this location. As a part of the reforestation so well done in this city, 300 000 pine trees were planted on the watershed. Nearly all the land on the watershed has been acquired, and all possible sources of pollution have been eliminated.

SOIL STUDIES FOR THE GRANVILLE DAM AT WESTFIELD, MASS.

BY CHARLES TERZAGHI.*

The Granville Dam forms part of the water supply system of Westfield, Mass. Messrs. Fay, Spofford and Thorndike, Boston, Mass., were the consulting engineers. The author was retained by this firm to investigate the permeability of the slopes of the reservoir, the foundation conditions and the materials available for dam construction. A detailed description of the dam itself and of the construction methods is found in the two preceeding papers of this JOURNAL. Hence the dam is supposed to be known to the reader and this paper will be strictly confined to a discussion of the soil problems.

The questions to be answered were as follows:

1. Will it be possible to find in the vicinity of the proposed dam site a sufficient quantity of material suitable for dam-construction purposes?
2. What will be the approximate quantity of water escaping through the dam?
3. What tests shall be made during construction of the dam?
4. What will be the approximate quantity of water escaping through the south slope of the proposed reservoir towards the adjoining valley?

The answers to these questions are without any interest to the general reader. Of broad interest, however, are the methods we used in our attempt to answer the questions and the lessons we learned while engaged in our investigations. Hence the scope of the paper is further limited to these two topics.

DESCRIPTION OF THE DAM SITE.

The site of the Granville Dam is located at the junction of Hollister and Tillotson brooks about 3 000 ft. above the point where Tillotson Brook joins Dickinson Brook to form Munn Brook. About 700 ft. upstream of the mouth of Tillotson Brook there exists a minor dam of the Westfield water supply, creating Winchell Reservoir. (Fig. 1.)

The valleys of Hollister and Tillotson brooks were formed by the brooks gradually cutting into a delta deposit of pleistocene and post-pleistocene age, formed in a lake that filled the space between Sweetman Mountain in the west, the valley of the Westfield Little River in the north, an unnamed mountain in the east and a terminal moraine located north of Granville in the south. In 1914 and 1915 the late Professor W. O. Crosby made a study of the geology of the valleys of Tillotson Brook and Hollister

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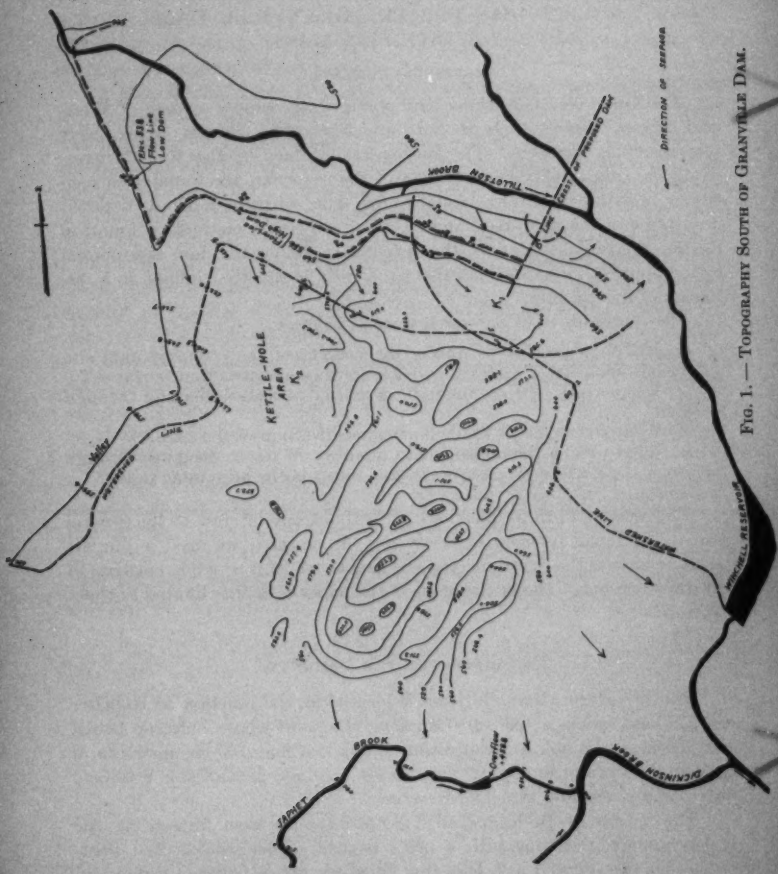

FIG. 1. — TOPOGRAPHY SOUTH OF GRANVILLE DAM.

Brook. According to his conception, the origin of the unconsolidated deposits that occupy the vicinity of the valleys was as follows:

In the basin thus formed (north of the Granville terminal moraine) was deposited a vast amount of gravel and sand (washed or modified drift), across and through which the modern brooks now wind. The drainage being obstructed to the southward, the streams found an outlet to the eastward through the deep notch at the north of the Sodom Mountain. Thus Munn Brook came into existence; and during post-glacial times the brook has cut out the deep and rugged gorge in the bottom of the notch, converting it into a miniature canyon.

The glacial sand and gravel, having been deposited along the margin of the waning ice sheet, over and among detached and partly detached masses of ice, developed, as the buried and half-buried ice melted, a very uneven surface, made up of knobs (kames), hollows (kettles) and winding ridges.*

In addition to this, Professor Crosby called attention to the existence of an esker on the west slope of Hollister Valley, about half a mile above the confluence of Hollister and Tillotson brooks.

The esker consists, in large part at least, of very coarse, waterworn gravel, and it is probable that this large body of open gravel would make a tight reservoir an economic impossibility.

Finally, concerning the proposed reservoir at the confluence of Tillotson and Hollister brooks, Professor Crosby pointed out:

On the southwest slope (right bank) — above 20 ft., the bedrock surface seems to slope away to the southwest, and this may be its maximum elevation on this side of the gorge. One of the borings shows, however, that the boulder clay continues to a height of at least 40 ft., and it probably goes higher, for the boulders on the surface indicate, as previously noted, that the boulder clay rises at least to the full height of the dam. There can be no question that the southwest wall of the proposed reservoir, northwest of the dam site, consists of sand and gravel down to the level of the meadow and possibly lower. This is the great deposit of washed or modified drift filling the wide and deep pre-glacial vally of a larger stream than Tillotson Brook, and it is just here where we encounter the most doubtful and uncertain feature of the entire project. (Pages A-9 and A-10 *loc. cit.*)

According to the topographic map the escaping water would ultimately find its way into Dickinson Brook and so be lost to the Westfield water supply. Fortunately, however, Japhet Brook intervenes. That water escaping from the proposed reservoir by the route indicated would be intercepted by Japhet Brook and turned into the intake reservoir is practically certain.

Further exploration of the dam site and its vicinity fully confirmed Professor Crosby's general conception of the geological situation. Concerning the details of the geology, however, two new facts were brought into

*Appendix A to the Preliminary Report of Fay, Spofford and Thorndike to the Board of Public Works, Westfield, Mass., p. 2 A.

evidence, facts that could not be predicted from the scanty data at the disposal of Professor Crosby. These facts relate to the character of the slopes at the dam site and the seepage from the reservoir through the modified drift towards the valley of Dickinson Brook.

The boulder clay supposed to form the right slope of the valley at the dam site does not exist. The strata throughout are delta deposits. These consist of two essentially different materials: A grey or greenish silty sand, evidently derived from decomposed granite, and a soft, reddish sand, rich in mica, similar in color to Longmeadow Sandstone. Since the material gets finer from the west towards the east, it seems that the main line of sand and gravel transportation extended from the north towards the south, across the center of the plateau west of the dam site. The filling-up of the glacial lake proceeded from this line towards the east. Due to continuous shifting of the current, the composition of the deposit is extremely erratic, and it is very difficult to predict how the different strata may change beyond the zone explored by test pits and test borings. In general, the permeability of the strata increases with the elevation, and in addition it increases from the east towards the west. However, these rules merely govern the broadest features of geological structure. As far as the details are concerned, the deposit consists of a most intricate succession of very permeable and almost impermeable strata. It may be considered as a dump of silty modified drift including a network of subterranean veins of more permeable sand and gravel. Thus on one of our trips to the dam site we noted, near the confluence of Dickinson and Tillotson brooks, at an elevation of about 420 ft., a pit in very coarse sand and gravel. This fact indicates the possibility that veins of coarse sand and gravel may extend all the way from the confluence of the two brooks up to the slope facing the south end of the Hollister esker, for if the vein is followed upstream, the material becomes of necessity coarser and not finer. Several similar veins may exist that are not open to inspection, and on filling the reservoir, this system of veins will change into an underground drainage system, carrying well-defined streamlets of seepage water. Since, however, the cross section of the coarse-grained veins undoubtedly decreases from the north to the south, so that, in the vicinity of Dickinson Brook, the cross section of the veins very likely amounts to a much smaller fraction of the total cross-sectional area than it does at the site of the reservoir.

This conclusion is confirmed by the observation that in the southern part of the kettle-hole area, most of the exposures disclose fine or medium stratified sand, while in the northern part, the coarse-grained strata predominate. One such deposit of fine sand was noted in the vicinity of the bridge across Japhet Brook, and several others along the old and the new road leading along the right slope of the Tillotson Valley northward towards the dam site.

Concerning the seepage from the reservoir towards Dickinson Brook it seems, after examination of the site, that there is very little hope of re-

covering an appreciable quantity of seepage water by way of Japhet Brook. In order to obtain more positive information concerning this important point, a rough stadia survey was made of the kettle-hole area and of the course of Japhet Brook. The results of this survey are shown in Figure 1. They disclosed the following facts: the kettle-hole area extends almost as far east as the upper rim of the right slope of the Tillotson Valley, and the overflow at Japhet Brook is located almost 50 ft. above the base of this area. Since a vein of very permeable material crops out at an elevation of about 420 ft. (the gravel deposit mentioned above) there is no chance that an appreciable part of the seepage may be recovered by way of Japhet Brook.

While surveying the kettle-hole area it was noted that several of the kettle holes are swampy, which indicates that their bottom is almost impermeable. In addition to this it was found that Japhet Brook flows for a distance of almost 5 000 ft. over the surface of the modified drift apparently without any loss of water. However, considering that the delta consists of a succession of almost impermeable strata and of very permeable veins, it is not surprising to find that certain sections of the kettle-hole area are underlaid by impermeable strata. Unfortunately the existence of such conditions has but very little bearing on the water-carrying capacity of the area as a whole, because the escape of the impounded water will certainly not have the character of continuous ground water movement but of flow through well-defined channels of limited dimensions.

Generally speaking, the dam site seems to be located in that section of the area which, from a geological point of view, is the most complicated of all. North of the dam site, in the adjoining part of the valley of Hollister Brook, the unconsolidated deposits are rather neatly divided into an esker on the right side of the valley and a terrace consisting of lake deposits on the left side. The esker seems to rest on a gentle slope consisting of boulder clay. On the other hand, the section located west of the little ravine entering from the south into the Tillotson Valley near the west end of the reservoir (see Fig. 1) is conspicuous by the absence of kettle holes; so much so that at the beginning it was believed that it consisted of boulder clay. By digging a test pit we learned, however, that this section, too, includes at least local deposits of modified drift.

In the light of these facts, the geological history of the deposits located at the dam site seems to have been as follows: The major part of the modified drift located west of the dam site was deposited in a lake with a surface elevation of about 640 ft. At this time, the ice still occupied the northern part of Tillotson Valley and part of the waters that transported the materials came from underneath the ice. The erratic delta deposit thus formed occupied the western part of the glacial lake. Then, while the ice withdrew towards the north, the remainder of the space occupied by the lake was filled up with finer and more uniform material (soft, reddish sand).

Exploration of the Dam Site.

The exploration of the dam site by test pits and test borings was started as early as 1914. The early explorations included: 8 test pits, located on a line parallel to the center line of the dam, about 100 ft. farther upstream. In addition to this, wash borings were made at 50-ft. intervals along the axis of the dam, and finally some percolation tests were executed for the purpose of testing the permeability of both the underground deposits and the available dam-construction materials. However, these investigations did not furnish more than very crude information concerning the underground conditions.

Since 1924 twenty-three wash borings have been made by B. F. Smith and Co. of Boston. Although attempts were made to secure "dry samples" it soon became evident that the information furnished by the test borings would not be sufficient for forming an opinion of the average permeability of the slopes of the reservoir. First of all, due to the presence in the ground of big boulders, very few test borings reached bed rock; second, no dry samples could be obtained from the strata of cohesionless or feebly-cohesive sand and gravel. Since the permeability of the deposit depends essentially on the permeability of the streaks and veins of sand and gravel, the test-boring method could not fail to be unsatisfactory. Hence, it was decided to supplement the test borings by test pits arranged along lines of steepest grade in such a manner that the bottom of each test pit was located somewhat below the upper rim of its neighbor on the valley side. Each test pit was made as deep as possible without timbering. In accordance with this program, 35 test pits were dug, representing 13 profiles. Of these profiles, 8 were located on the south slope and 5 on the northeast slope of the proposed reservoir. The distance between the profiles ranged from 200 to 500 ft. The test pits located on the south slope served for investigating the permeability of the deposits, because the danger of serious seepage losses was confined to this slope. (See Fig. 1.) The test pits located on the northeast slope were made essentially for the purpose of selecting the most favorable site for borrow pits.

In the test pits, samples were taken of each well-defined stratum (about three pounds of each). These samples were placed in small canvas bags and shipped to the author's laboratory for the purpose of being tested. Thus the author received, during the first part of 1926, 43 samples obtained by test borings (in small glass containers with metal screw caps) and 94 samples collected from the test pits (in bags or boxes). Since most of the samples extracted from the drill holes were washed, the samples were merely fit for giving a general conception of the nature of the strata, and soil tests were confined to the 94 samples obtained from test pits.

Soil Tests.

The chief object of the soil tests was to establish a basis for estimating the magnitude of water losses due to percolation around the proposed dam and for judging the suitability of the available materials for dam-construction purposes.

Due to the exceedingly variable character of the materials, much information was required for estimating the average permeability of the ground. To obtain this information two different methods were available: (1) the direct determination of the coefficient of permeability of the samples by means of permeability tests or (2) the computation of the coefficients of permeability from the results of mechanical analyses. Discouraging experiences with primitive permeameters (great errors due to filter-skin and air content) on the one hand and lack of reliable percolation formulae on the other led us to combine the two methods in the following manner: We determined the coefficients of permeability with utmost care for 8 representative samples, investigated for each one of these samples the relation between effective size and the coefficient of permeability, divided the remaining 86 samples into groups corresponding to the representative samples and computed for the members of each group the coefficient of permeability from the results of a mechanical analysis and from the information obtained by the permeability tests of the representative samples. Since the permeability of the same sand may be very different, depending on the volume of voids, we were also obliged to determine for each material the volume of voids in the densest and in the loosest state, and to introduce the volume of voids as an additional variable into our equations. The following represents a brief description of our testing methods.

Mechanical Analysis. Since most of the samples contained appreciable quantities of silt, the mechanical analysis had to be performed by combined sieve-analysis and elutriation. Those samples which required partial elutriation were first washed in an evaporating dish, thus separating the coarser constituents from the finer. The finer fractions were brought into suspension, in concentrations of approximately 30 grams of dry material per liter of water. In the elutriation tanks the material was allowed to settle through a vertical distance of 10 cm. for periods of 7½ minutes and 1 hour. The turbid liquid siphoned out of the tanks, after these periods, contained in suspension the particles with a grain-size of 0.02 to 0.006 mm. (coarse silt) and of less than 0.006 mm. (fine silt and clay) respectively. In order to wash out of a sample all the particles of a given size, the sedimentation process had to be repeated from 8 to 10 times, for each size. The weight of each fraction was determined after drying it for 5 hours at 105° C. in a thermostat-controlled oven. The sieve-analysis was performed with 200-gram samples by means of a set of Tyler standard sieves.

Specific Gravity, Volume of Voids and Voids-ratio. The data concerning the volume of voids were based on the dry weight of the material contained in a steel cylinder with a height of 8 in. and an internal diameter of 2 in. The sample was first placed in the cylinder in 2-cm. layers, each

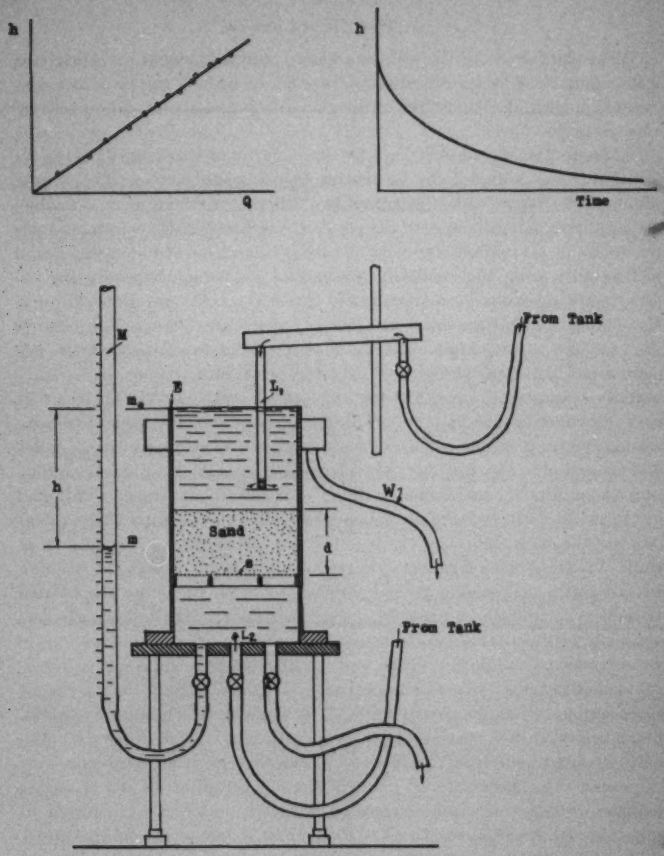

FIG. 2. — SKETCH OF PERMEAMETER AND DIAGRAMS OF DISCHARGE MEASUREMENTS (upper left) AND STANDPIPE OBSERVATIONS (upper right).

layer being thoroughly compacted by means of a steel pestle. Then the cylinder was emptied and filled once more, the material being poured loosely into the vessel.

In order to compute the volume of voids, it is necessary to know the specific gravity of the dry material. Since the specific gravity was found to vary within very wide limits (2.68 to 3.05) it was necessary to determine it for several typical samples (13 tests) and to estimate its value for the others. The specific gravity was determined by means of 40-gram samples in a

100-cc. pycnometer bottle. Suspensions of silty materials were boiled prior to measuring the weight of the pycnometer in order to eliminate the error due to the pressure of air bubbles.

From the specific gravity of the grains, the weight of the content of the cylinder, and the volume of the cylinder, the volume of voids and the voids-ratio of the samples were computed. The term "volume of voids" indicates the ratio between the volume of the voids and the total volume occupied by the sample. However, for various reasons, it is preferable to express the density of the samples not by the volume of voids, but by the "voids-ratio," *i.e.*, by the ratio of the volume of voids to the volume occupied by the grains.

Coefficient of Permeability. The coefficient of permeability was determined by means of a permeameter with a cross-sectional area of 160 sq. cm., on samples with a dry weight of approximately 2 000 grams. In the permeameter, the samples rested on a coarse-grained filter-layer, supported by a horizontal screen. (Marked S in Fig. 2.) While placing the samples in the container, care was taken to remove the air from the voids of the samples. For this purpose, each sample was thoroughly wetted in an evaporating dish prior to filling the container, and the water level was always kept a fraction of an inch above the surface of the sample during the filling of the container. After the container was filled, the water was allowed to rise slowly to the crest of the overflow (E) and then to waste away through the discharge pipe (W) until the discharge became clear. This was necessary to prevent the formation of a filter skin during the test.

For measuring the permeability of coarse-grained samples, the water flowing through the sample under various hydrostatic pressures was measured by collecting the discharge in a graduated cylinder. (Method of testing: Measuring discharge.) The permeability of silty samples was computed from the speed with which the water rose in a calibrated standpipe (M) communicating with the space located below the filter. (Method of testing: Standpipe observation.) In each test the hydraulic gradient was first gradually raised from 0.1 up to about 3.0, the water entering the container through feed pipe (L_1) and flowing downward through the sample. Then the direction of flow was reversed, feed pipe (L_2) being used, and the hydraulic gradient was raised from 0.1 up to 0.7. Finally, the direction of flow was reversed once more for the purpose of obtaining a check on the first run. If, in spite of the care exercised in filling the container, a filter-skin is formed, the coefficient of permeability computed from the results of the first and third run will prove to be considerably smaller than that computed from the results of the second run. However, this never occurred with the method employed by us in filling the container.

Above the permeameter in Figure 2 are shown the results of observations made while running the test by measuring the discharge (Q) and by timing the rise of water (h) in the standpipe. In all the tests made by measuring the discharge, its value (Q) was found to increase almost strictly in direct proportion to the hydraulic head (h_1); this means that the points as shown in Figure 2 fall on a straight line.

Test Results.

The results of the mechanical analyses were plotted such that the abscissae (x) represent the grain-size (d) on a logarithmic scale ($x = \log d$ and $x = 0$ corresponding to a grain-size of 1 mm.) and the ordinates (y) represent the percentage weight of that fraction whose grain-size is smaller than the value indicated by the corresponding abscissa. As defined by Allen

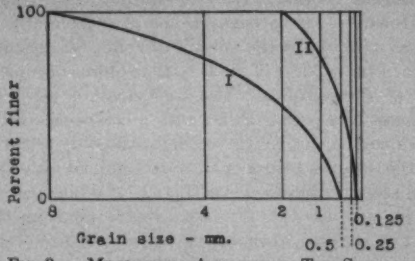

Fig. 3. — Mechanical Analysis of Two Soils. — Grain-size plotted on arithmetic scale.

Hazen that point on the grain-size curve whose ordinate equals 10 per cent. represents the "effective size" of the material, and the ratio between the grain-sizes that correspond to ordinate of 60 and 10 per cent. respectively, is the "uniformity coefficient." The greater the uniformity coefficient, the less uniform is the material.

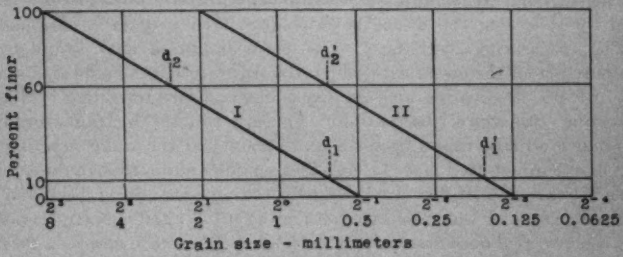

Fig. 4. — Mechanical Analysis of Two Soils. — Grain-size plotted on logarithmic scale.

The advantage of using a logarithmic scale is shown in Figures 3 and 4. Figure 3 represents the grain-size curves for two uniformly graded materials I and II with equal uniformity coefficients but with different effective sizes, the grain-sizes being plotted on a natural scale. The grain-size curves are very different, and for the finer material (II) it is obviously difficult to determine accurately the effective size. Figure 4 shows the grain-size curves for the same materials I and II with the grain-sizes plotted on a logarithmic scale. Since the two materials are supposed to be uniformly

graded and to have equal uniformity coefficients, it is easy to prove that their grain-size curves must be straight, parallel lines. Decrease of the uniformity coefficient causes the grain-size curve to become steeper, irrespective of effective size, and the shape of the curves indicates the degree of uniformity of grading. Hence by merely glancing at a diagram of the type shown in Figure 4, we can learn several things that are obscured in Figure 3 by the method of presentation.

Figure 5 represents the results of a mechanical analysis and of a per-

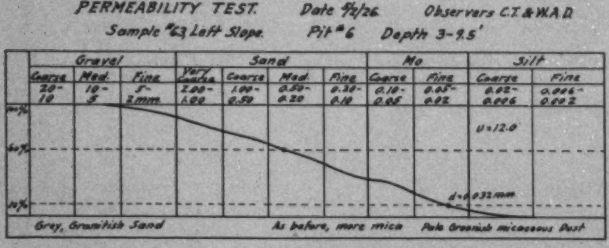

FIG. 5. — RESULTS OF MECHANICAL ANALYSIS AND PERMEABILITY TEST.

meability test. Inclusion of a description of the different fractions of the material was necessary in order to correlate them with the samples whose permeability had previously been determined. The results of the mechanical analysis of samples secured from four profiles on the south slope of the reservoir are shown in Table 1 and demonstrate how exceedingly variable are the underground conditions encountered at the reservoir site.

The next step consisted in determining, for the different soil samples, the volume of voids in a loose and in a compacted state, by means of the method described before. At the outset we hoped to be able to use a short cut by establishing some relation between the extreme volumes of voids (loose and dense) and the uniformity coefficient, but it was soon found that no such relation seems to exist. This can be seen in Table 2 which gives the uniformity coefficient and the extreme voids-ratios. In general, for materials with a small effective size (0.01 to 0.05 mm.) the difference between the maximum and the minimum volume of voids is greater than for coarser materials (effective size >0.1 mm.), and the minimum volume of voids is somewhat higher for uniform materials (uniformity coefficient <5) than for non-uniform materials (uniformity coefficient >15); but these are merely statistical relations that cannot be utilized when dealing with specific cases.

TABLE 1. — MECHANICAL ANALYSIS OF SOIL SAMPLES FROM SOUTH SLOPE OF TILLOTSON VALLEY.

Profile No.	Pit No.	Sample No.	Depth of Pit. ft.	Elevation. ft.	Effective Size. mm.	Uniformity Coefficient.
V	8	41	3–12	554.8–545.8	0.29	2.9
	8	42	21	536.8	0.27	5.5
	4	45	3–8	527–522	0.15	5.3
	4	46	8–18.7	522–512	0.09	2.2
	4	47	18.7	512	0.02	10.4
	7	48	3–6	509.4–506.4	0.64	8.6
	7	49	11–12	501.4–500.4	0.15	5.3
	7	51	12–	500.4–	0.06	6.6
VI	3	52	2	544.4	0.60	41.4
	3	53	5	541.4	0.49	9.1
	3	54	13	533.4	0.15	2.4
	10	37	1–8	543–536	0.61	7.5
	10	38	15	529	0.55	61.0
	10	39	17	527	<0.002	>100
	10	40	20	524	0.10	2.7
VII	1	55	6	530.7	0.34	6.0
	1	56	13	523.7	0.22	3.6
	2	57	10	504.4	0.13	4.4
VIII	9	60	0–11	538.0–527.0	0.60	41.4
	9	61	11–15.5	527.0–522.5	0.15	4.0
	9	62	15.5	522.5	0.08	22.5
	20	58	0–6	....	0.27	7.2
	20	59	6–16	....	0.19	>50

TABLE 2. — UNIFORMITY COEFFICIENTS AND VOIDS-RATIOS OF LOOSE AND COMPACTED MODIFIED DRIFT.

Sample No.	Uniformity Coefficient.	Voids-ratio.		Sample No.	Uniformity Coefficient.	Voids-ratio.	
		Compacted.	Loose.			Compacted.	Loose.
5	3.3	0.53	0.84	19	8.9	0.63	1.07
34	3.6	0.44	0.76	30	9.2	0.58	1.00
7	4.5	0.78	>1.4	35	10.5	0.42	0.75
36	5.4	0.54	1.32	28	11.8	0.59	1.12
22	5.6	0.47	0.82	4	12.1	0.37	0.77
20	6.7	0.64	1.12	1	12.9	0.38	0.77
32	7.0	0.58	1.09	2	14.3	0.95	>1.4
29	7.1	0.37	0.64	23	15.7	0.44	0.73
21	8.4	0.58	1.07	6	16.8	0.51	0.87

Mechanical analyses and density determinations were expected to furnish the data for computing the coefficient of permeability. Since the time when Slichter published his investigations, it has been known that the coefficient of permeability of a sand decreases quite rapidly with decreasing volume of voids. Figure 6 shows the relation that exists between the voids-ratio and the permeability for sands according to an equation derived by the author. There is, however, very little difference between

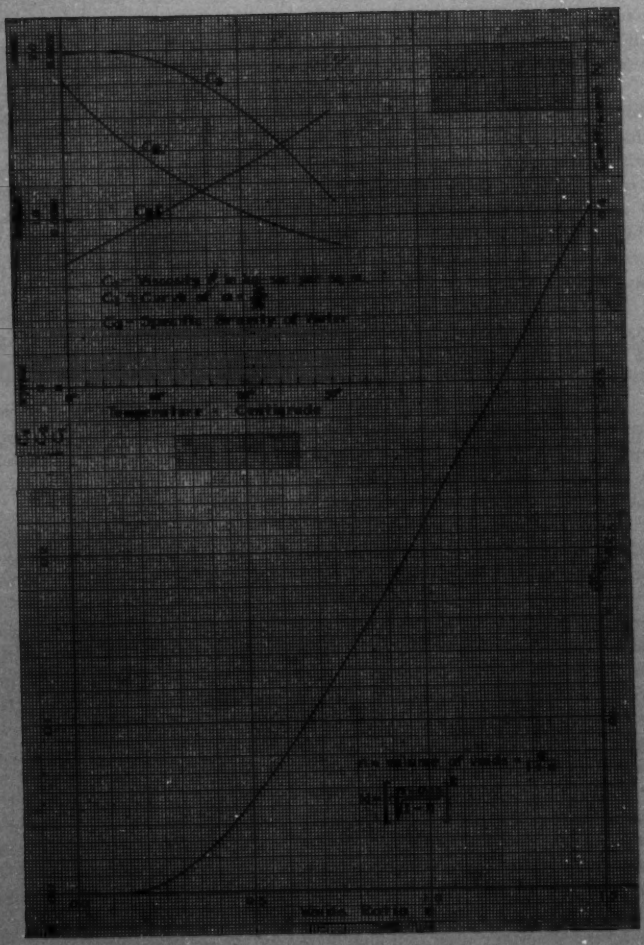

FIG. 6. — RELATION BETWEEN VOIDS-RATIO AND PERMEABILITY OF SANDS.

the numerical values furnished by the complicated Slichter equation and the simple relation derived by the author. For clays the permeability-voids-ratio curve is very much steeper, and for micaceous sands or sands with an appreciable silt content it lies between the two extremes, its shape depending on the character and the percentage of the admixture.

Considering the importance of the effect of the voids-ratio on permeability, it is obvious that we cannot compare the permeability of sands or silts with each other, unless we reduce the data to identical conditions, not only with respect to temperature but also with respect to voids-ratio. Hence it was decided to reduce the data to a standard voids-ratio of 1.00 (volume of voids 50 per cent.) and a standard temperature of 15° C. (reduced coefficient of permeability). The reduction was performed by means of the curves shown in Figure 6, wherein the effect of the silt and mica content on the shape of the voids-ratio-permeability curve was neglected.

The reduced coefficients of permeability (k) of the representative samples was found to range between 188 d^2 for sands with little silt and mica, and 5.2 d^2 for sands with a high mica and silt content, d being the effective size in cm.

On account of the wide margin between these two limits, the fractions furnished by mechanical analysis had to be very carefully examined, and it required much time and labor until we had gained the experience needed for properly classifying our samples according to their relationship to the standard samples mentioned at the outset of this section.

The great variations in grain-size and in uniformity shown in Figures 7 and 8, are obviously associated with similar variations in the mineralogical composition of the materials. This can be learned from the results obtained by Professor W. H. Newhouse for two representative samples. Both samples came from the borrow-pit area, on the northeast slope of the reservoir. One of them was a "grey hardpan" with an effective size of 0.029 mm. and a uniformity coefficient of 16.0, representing the upper stratum in the profile (Fig. 8) and the other a "soft, reddish, uniform sand," with an effective size of 0.021 mm. and a uniformity coefficient of 3.95, representing the lower stratum in the same profile. These samples were first of all split up by mechanical analysis into fractions with different fineness; then each fraction was examined by Professor Newhouse under a high-power, petrographic microscope. This investigation gave information on the degree of angularity and the size of the grains. The identification of the minerals was checked with the petrographic microscope, and the quantitative composition of the fractions was determined by counting the grains of different minerals present in the fraction. For the fraction passing the 200-mesh screen, the counting was confined to grains larger than 0.02 mm. The results of this study are shown in Table 3.

In both samples the feldspar was found to be more or less decomposed.

It should be noted that the mineralogical composition of each sample varies widely from fraction to fraction. The angularity of the grains in-

TABLE 3. — RESULTS OF PETROGRAPHIC INVESTIGATION OF TWO SAMPLES BY PROFESSOR W. H. NEWHOUSE.*

Sieve Size. Meshes per inch.	Weight of fraction. Per cent. of total weight.	Granite.	Quartz.	Feldspar.	Mica.	Hornblende.	Garnet.	
			1. Grey, granitic sand.					
>10	16	43	42	5	..	5	5	Quartz angular, granite rounded.
10–20	11	5	79	7	3	2	4	Quartz angular, feldspar subangular, granite round to subangular.
20–35	7	..	80	3	14	3	2	Subangular to angular.
35–65	9	..	90	4	3	2	1	Quartz angular, hardly rounded.
65–150	15	..	79	..	12	3	6	Quartz angular.
150–200	19	..	66	14	8	..	12	Very angular. Splintery grains.
200–0.02	23	..	81	11	..	..	8	Quartz angular.
			2. Soft, reddish uniform sand.					
>14	7	30	50	4	8	3	5	Quartz subangular to angular, granite round to subangular.
14–28	4	..	70	14	9	4	3	Quartz very angular, very little abraded, none splintery.
28–48	8	..	57	18	20	3	2	Quartz subangular, feldspar rounded.
48–100	11	..	40	15	43	1	1	Quartz angular, feldspar subangular.
100–200	12	..	72	18	5	3	2	Quartz angular, feldspar partly rounded.
200–0.02	58	..	77	14	..	..	9	Quartz angular, feldspar rounded.

*Massachusetts Institute of Technology, Cambridge, Mass.

dicates that the material was transported by flowing water through a short distance only. The two samples represent fair averages of the two main types of fine-grained deposits. There were, however, some local accumulations that consisted almost entirely of mica and others that had an extremely high content of garnet. In brief, the deposit was found to contain an extraordinary variety of non-consolidated or half-consolidated accumulations of grains.

STRUCTURE AND PERMEABILITY PROFILES OF THE RESERVOIR SLOPES.

The soil tests furnish data upon which can be based an opinion on the geological and technical characteristics of the dam site. A study of the geological characteristics is imperative, because the soil data merely inform us about the materials obtained from a few individual profiles. In order to learn something about the conditions existing between the profiles we are obliged to supplement our findings by geological evidence.

The soil tests reveal the fact that both the uniformity coefficient and the effective size of the deposits are extemely variable. It is known that sedimentation in quiet water leads to the formation of sediments with low uniformity coefficients (1 to 5), while high uniformity coefficients are characteristic of sediments deposited in rapidly flowing water. Hence variations

in uniformity and permeability reveal some of the most salient features of the geological history of any deposit. In order to bring these features into prominence, the uniformity and permeability graphs for both the northeast and the south slope of the proposed reservoir were plotted. Table 4 shows the information upon which the graph for the south slope of the site was based.

A study of this table demonstrates that both the intensity and direction of the current varied within the section located on both sides of the proposed dam axis to a very considerable extent. There seems to be no definite relation between the uniformity and coarseness of the various layers although, in general, the rapid-current deposits are far less permeable than the uniform ones.

Due to the continuous variation in the intensity and direction of the currents, the geological structure of the south slope is very complex. This can be learned from Figure 7 which shows the structure of the south slope in the vicinity of the dam axis. The quiet-water and slow-current deposits form bars of lens-like cross section, surrounded by deposits laid in more rapidly flowing water. The permeable strata of this section are, in general, very similar to one another, namely, grey granitic sand with some light-colored mica. As accessory constituents there occur snow-white grains of decomposed feldspar and dark brown grains of garnet. The less permeable strata are distinguished by containing a high percentage of yellow silt. There is only one essentially foreign deposit within this system. This consists of a layer of remarkably uniform, soft, reddish sand, encountered in one of the test pits between elevation 556.5 and 540.0. This material seems to be fairly identical with the reddish sand that, according to Figure 8, forms the major part of the slope on the northeast side of the proposed dam.

Proceeding along the southern slope of the valley from the dam site towards the west, the less permeable strata disappear, and the slope consists essentially of very permeable, fairly uniform sands and gravels. These materials have been deposited not far from the main line of transportation. Similar materials were encountered in a test pit on the promontory between Tillotson and Hollister brooks, north of the proposed dam site.

Some difficulties were encountered in exploring the northeast slope of the reservoir site, due to the fact that this slope is covered with a layer of creep to an average thickness of 10 ft. The creep consists of grey, granitic sand and gravel, cemented together with a greenish, cement-like silt, very rich in mica. On some parts of the slope, the material is so dense that it was called hardpan in the test-pit records. Due to the remarkable density of the material, we doubted for some time that it had been transported by creep. However, the data furnished by several additional test-pits established this fact beyond any doubt. Underneath the blanket of creep, the underground consists of a soft, reddish, fairly uniform sand of very variable effective size, coarse-grained and fine-grained layers succeeding one another

TABLE 4. — PERMEABILITY AND UNIFORMITY OF SUBSOIL RIGHT SLOPE OF TILLOTSON VALLEY.

Profile No.	Distance from Axis of Dam.	Pit No.	Elevation. ft.	Permeability. cm. per sec. X10⁶.	Uniformity Coefficient.
I	300 ft. below	19	523	2.8	22
			513	0.03	14
			509	0.25	37
		21	507	1.7	37
			498	280	3
			488	1.5	43
			484	80	6
II	100 ft. below	11	557	2.6	5
			555	8.2	5
			548	40	3.2
			542	9	2.7
		12	538	5	5
			535	96	12
			528	0.9	1
		13	515	2.2	9.5
			510	18.0	22.5
		14	501	1.6	10.5
			495	200	8
III	100 ft. above	15	548	1.6	10
			535	2.2	8
		16	531	1.0	11
			523	2.0	11
			520	5.7	150
		17	515	0.5	15
			508	2.0	>68
			500	1.7	>54
		18	492	2.4	7
IV	265 ft. above	23	545	1.2	13
			541	14	13
			537	2.6	14
		5	527	1.2	14
		24	511	4.4	10
			506	0.01	103
			500	61	6
		25	495	4.4	19
V	495 ft. above	8	555	580	3
			546	360	5.5
		4	527	11	5.3
			522	61	2.2
		7	511	1	10.4
			509	1 350	8.6
			506	120	5.3
			500	15	6.6
VI	915 ft. above	3	544	650	41.4
			541	680	9.1
			533	170	2.4
VII	1 415 ft. above	1	531	540	6.0
			524	310	3.6
		2	504	80	4.4
VIII	1 915 ft. above	20	570	220	7.2
			565	65	>50
		9	538	650	41.4
			527	130	4.0
			523	11	22.5

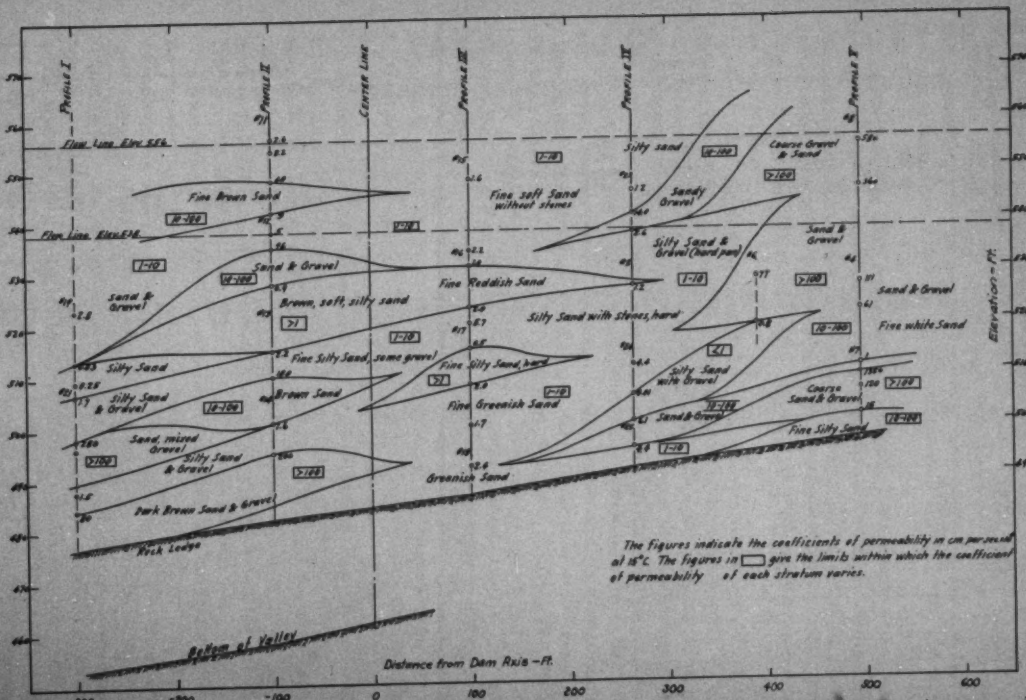

FIG. 7. — GEOLOGICAL PROFILE AND PERMEABILITY GRAPH OF SOUTH SLOPE OF RESERVOIR ADJACENT TO DAM AXIS.

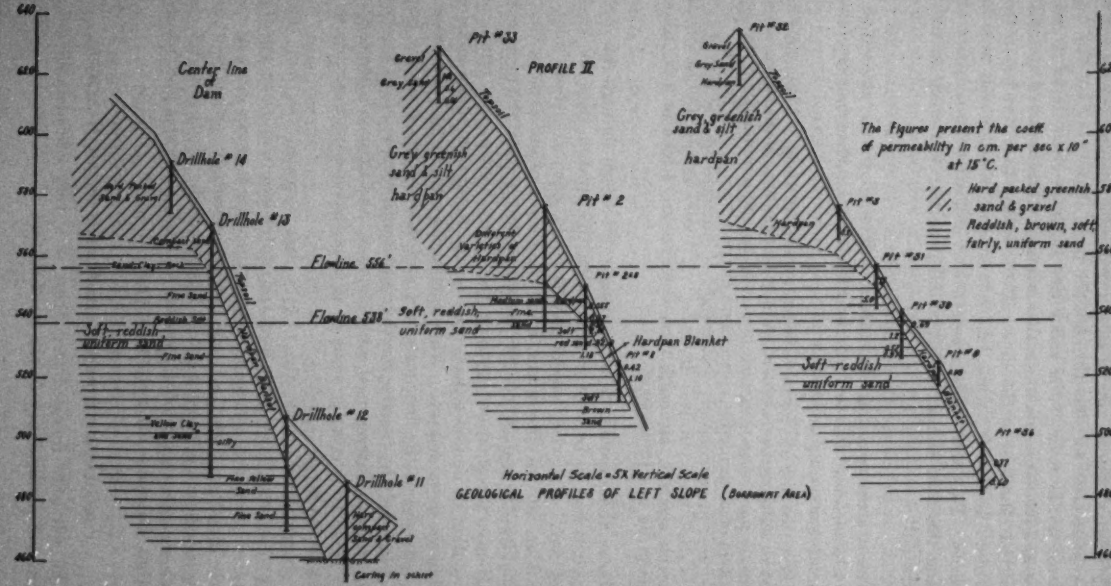

FIG. 8. — GEOLOGICAL PROFILES OF NORTHEASTERN SLOPE OF RESERVOIR (BORROW-PIT AREA).

in very short distances and in an unpredictable manner. Figure 8 shows three typical cross sections through the right slope.

Towards the floor of the valley the thickness of the layer of creep increases, and its composition is rather erratic. Thus in one of the drill holes located near the foot of the slope, all the wash water was lost between a depth of 12 and 25 ft., while below this depth the material was far less permeable. This and similar observations indicated the necessity of constructing the core wall so that it would penetrate the creep to the "fine yellow sand."

The stratum covering the bottom of the valley was explored as early as 1914 by means of two test pits, and Professor Crosby made the following statement concerning this stratum:

> These two pits are (or have been) 10 to 12 ft. deep. They show chiefly angular or subangular boulders from 1 to 4 or 5 ft. in diameter, as closely and tightly packed as they possibly could be and in a matrix of clayey sand and gravel of a decidedly impervious character . . . . That this boulder formation is approximately water-tight is indicated by the fact that when the pits have been pumped dry, they refill very slowly, by an almost imperceptible seepage, although their bottoms are 4 to 6 or 8 ft. below the surface of the nearby brooks.

Hence, there seemed to be no danger of seepage through this stratum. However, it had to be kept in mind, that this stratum does *not* consist of boulder clay. If it were boulder clay, it would continue on both sides of the valley floor, into the mountain, while the results of our explorations show that boulder clay is conspicuously absent on both sides of the proposed dam site. Hence, the stratum covering the valley floor merely represents an accumulation of boulders that, prior to the formation of the valley, were buried in the delta deposit. While cutting its way through the deposit, the brook removed all the finer materials, and the boulders gradually accumulated. In brief, the boulder stratum represents what is left of a very much thicker stratum, interspersed with boulders, and in such a deposit one is always likely to encounter veins or streaks that are more permeable than the rest of the layer.

Concerning bed rock, the test borings revealed the fact that the surface of the rock rises at the west end of the dam to an elevation of about 20 ft. above the valley floor, and beyond the ridge it slopes away in a southwesterly direction. The bed rock consists essentially of gneiss whose top layer seems, here and there, to be badly shattered. However, while drilling through shattered parts of the gneiss, no wash water was lost, which indicates that the seams are not continuous.

Percolation through the Slopes of the Reservoir.

The soil tests revealed that there is a very considerable difference between the permeability of the northeastern and southern slopes of the

reservoir. Although the deposit of fine reddish sand, which according to Figure 8 forms the bulk of the northeastern slope, contains a few layers of coarser material, there seems to be no reason to expect appreciable water losses through this slope, because the slope is covered with a continuous and almost impermeable blanket of hardpan. Any sloughing that may occur on account of local saturation can easily be remedied by local drainage.

Far less favorable are the conditions existing in the proximity of the south wing of the dam and along the south slope of the valley. Table 4 shows that the permeability of the slopes is by no means uniform, inasmuch as the deposit is composed of a succession of very permeable and very feebly permeable layers. For this reason seepage will undoubtedly assume the character of underground flow through fairly well-defined and comparatively narrow layers. However, for the purpose of estimating the loss due to seepage, it is necessary to disregard this fact and to base the computation on the average coefficient of permeability of the whole system. By multiplying the thickness of each stratum with its coefficient of permeability and dividing the sum of the products by the combined thickness of the layers, we obtained for the average coefficient of permeability of the south slope, along the various profiles, the following values:

Profile.	Samples.	Permeability k in cm. per sec.	Profile.	Samples.	Permeability k in cm. per sec.
I	1– 7	0.0028	V	37–51	0.0245
II	8–18	0.0026	VI	52–54	0.0362
III	19–27	0.0002	VII	55–57	0.0270
IV	28–36	0.0012	VIII	60–61	0.0500

This set of figures shows plainly that the permeability of the slope is a minimum in the vicinity of profile III and that the permeability increases from this profile downstream as well as upstream.

For estimating the quantity of seepage, one may divide the permeable area into two sections: One semicircular area (K_1) including the water veins flowing around the south wing of the dam and a rectangular area (K_2) extending from profile VI to profile IX towards the confluence of Dickinson and Tillotson brooks. (See Fig. 1.)

Concerning the seepage around the right wing, we made the following simplifying assumptions based on the results of the permeability tests: Within a distance of 100 ft. from the right wing including profiles II and III, the slope is practically impermeable. From a distance of 100 ft. to a distance of 650 ft. (profiles I, IV and V) the underground is permeable and its coefficient of permeability is equal to the average of the coefficients of permeability of the slope between profiles IV and V. This coefficient amounts to 0.013 cm. per sec. Since the flow line of the reservoir is at elevation 538, and rock ledge is located at elevation 477 (more or less), the total head under which the water enters the permeable slope amounts to 61 ft.

Basing our computation on these figures, we found that the loss of water through the semicircular area (K_1 in Fig. 1) may amount to as much

as 240 000 gal. per day. If the flow line is raised to elevation 556, the loss due to seepage will become 400 000 gal. per day.

In order to evaluate the quantity of seepage through the slope between profiles VI and VIII we assumed that from the reservoir towards Dickinson Brook, over a distance of approximately 3 000 ft., the surface of the rock is plane and has a uniform slope, from elevation 500 at the reservoir down to 410 at Dickinson Brook; and that the unconsolidated fill located above the rock has a uniform coefficient of permeability (k) equal to the average co-

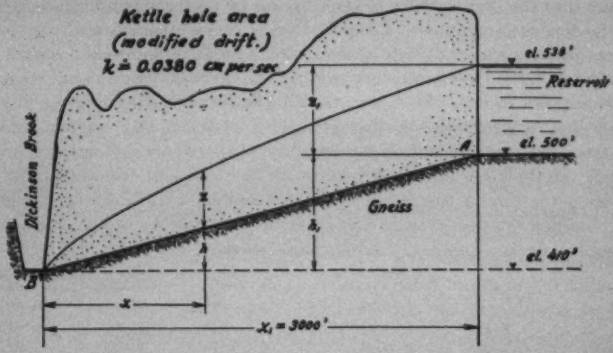

FIG. 9. — SEEPAGE FROM THE RESERVOIR TOWARDS DICKINSON BROOK.

efficient of permeability of the materials encountered in the test pits of profiles VI to VIII.

These fundamental assumptions are graphically represented in Figure 9. At the reservoir the elevation of the ground-water level will be 538. Between the reservoir and the valley of Dickinson Brook it will sink down to elevation 410. In order to compute the quantity of water flowing through the permeable layer, we had to compute first the shape of the ground-water surface, and then we had to find the relation existing between the coefficient of permeability and the topographical data on which the computations are based. Let (q) be the quantity of water flowing per unit of width through the permeable stratum, (z) the depth of the ground water at a distance (x) from the point of discharge (B), and $c = \frac{h_1}{x_1}$ the average slope of the surface of the impermeable bed rock.

Since the slope of the rock surface is gentle, the slope of the line of saturation is pretty nearly equal to $\frac{dz}{dx} + c$. Hence, if we accept for our computation Darcy's formula for the flow of water through permeable strata, we obtain

$$q = kz\left(\frac{dz}{dx} + c\right) \text{ whence } x = \frac{1}{c}\int_{z_0}^{z_1} \frac{zkc}{q - kzc}\, dz$$

By integrating this equation we obtain

$$x = -\frac{1}{c}\left(\frac{q}{kc}\log\frac{q-kcz}{q}+z\right) \text{ or } (z+cx)\frac{kc}{q} = -\log\left(1-\frac{kcz}{q}\right) \ldots\ldots (1)$$

This equation represents the solution of our problem.

The average coefficient of permeability of the materials encountered in the test pits located on profiles VI, VII and VIII is:

$$k = \tfrac{1}{3}\,(0.0362+0.0270+0.0500) = 0.038 \text{ cm. per sec.}$$

If we introduce this value into equation (1) and assume in addition that the width of the belt through which the water escapes is approximately 1 200 ft., we obtain the following results:

Flow line at elevation 530: Loss due to seepage = 1.46 m.g.d.
Flow line at elevation 556: Loss due to seepage = 1.80 m.g.d.

It has been pointed out that the escape of impounded water will undoubtedly occur in the shape of water veins circulating through layers of very permeable material, which will come to the surface as springs on the slope of Dickinson Valley. In our calculations we assumed that the coefficient of permeability of the delta deposit is uniform throughout. This means that the permeable layers have the same width and permeability throughout. Since it is very probable that both the width and permeability of the coarse-grained strata decrease towards the rim of the delta, *i.e.*, from the reservoir towards Dickinson Brook, the results of our computation may be much too high. Nevertheless, there is the possibility of seepage losses amounting to as much as 1 m.g.d. The permeable part of the slope has a length of approximately 2 000 ft. and an area of approximately 500 000 sq. ft. In order to stop the leakage it would be necessary to provide the whole area with a water-tight lining. Hence, it was recommended to take into account the fact that the storage project *may* involve the necessity of lining the area under consideration. But since there is also a possibility that leakage will be considerably smaller than indicated by the computations, the placing of the lining should be postponed until the dam is constructed and we are in a position to determine by direct measurement the loss due to seepage. This determination will be made by means of measuring weirs that were established across both Tillotson and Hollister brooks, beyond the flow line of the reservoir, and by keeping records of the amount of water entering the reservoir, the quantity of water fed into the water supply system and the fluctuations of water level in the reservoir.

Seepage Through the Dam.

Since the report on the Granville Dam was prepared, the cross section of the dam has been modified in that the upstream slope was flattened and the downstream slope was steepened. The following discussion refers to the original cross section.

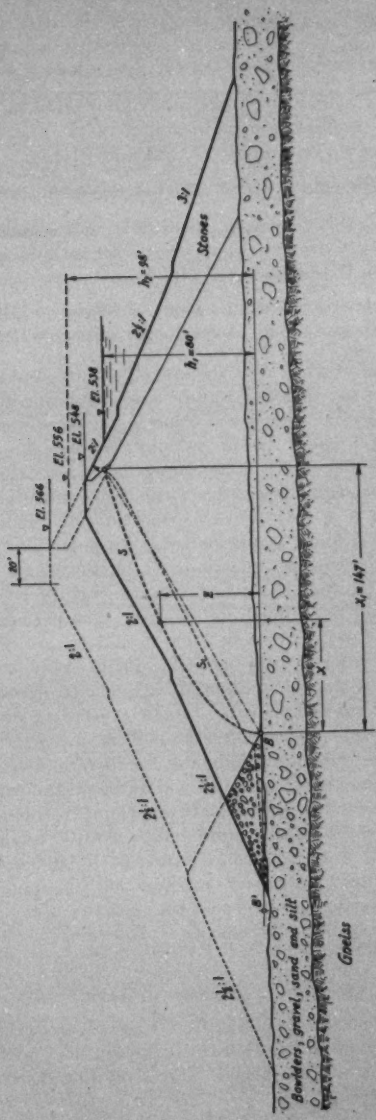

FIG. 10. — SATURATION LINE OF THE DAM.

Among the materials encountered in the proposed borrow-pit area on the northeastern slope of the valley, between the dam axis and profile V, two essentially different types could be distinguished: A reddish, fairly, uniform, micaceous sand, and a grey or greenish, very mixed-grained material, classified as "dense packed grey sand," "grey hardpan," "clay hardpan," and the like. (See Fig. 8.) Due to its uniformity and the impossibility of thoroughly compacting it, the reddish sand represents by no means an ideal dam-construction material. On the other hand, the "hardpan" possesses all the properties of a good dam-construction material. The average volume of voids of the samples was found to be 30 per cent. in the compacted state against 40 per cent. for the compacted samples of reddish sand. Hence, the material is very stable. Its coefficient of permeability is low, and, owing to the high percentage of coarse constituents, the volume change produced by saturation is negligible. Hence, it was obvious that the dam-construction material should be taken from the "hardpan" deposits. From test-pit evidence it was estimated that one could secure an ample supply of "hardpan" with a coefficient of permeability of less than 0.0001 cm. per sec. The following computations are therefore based on this figure.

Figure 10 shows a simplified cross section through the dam according to the original project. From the hydraulic point of view the most important dimensions of the dam are: The horizontal distance (x_1) between the two ends (A and B) of the saturation line and the difference (h_1) in elevation between these two points. For a flow line located at elevation 538, $h_1 = 80$ ft. and $x_1 = 147$ ft.

If we accept the traditional assumption of Chézy that the velocity of flow of seepage water in every vertical cross section is equal to the hydraulic gradient of the line of saturation in the same vertical, the relation between the coefficient of permeability (k), the quantity of seepage (q) per unit length of dam and the hydraulic gradient $\frac{dz}{dx}$ will be: $q = kz \frac{dz}{dx}$

By integrating this equation we obtain:

$$q = k \cdot \frac{h_1}{2} \cdot \frac{h_1}{x_1} \qquad \ldots\ldots\ldots\ldots\ldots (2)$$

If we introduce into this equation the value $k = 0.0001$ cm. per sec., and the values of $h_1 = 80$ ft. and $x_1 = 147$ ft., we find that the loss of water due to seepage across the dam may amount to 46.5 gal. daily for each foot of length of the highest part of the dam.

Extending our computations to the full length of the dam and considering that the width of the dam decreases with increasing elevation above its base, we find that the loss due to seepage across the dam may amount to $46.5 \times 560 = 25\,500$ gal. per day.

There is no doubt that there is considerable difference between the assumptions on which this traditional method of computation is based,

and the conditions that actually exist in a dam. At present, however, we have no better method for computing the magnitude of percolation. Due to disregarding the existence of the dam-construction material located to the right of point (A) in Figure 10, the values furnished by formula (2) are undoubtedly too high. Hence, it is believed that the errors are on the safe side.

If the height of dam is increased without previously removing the permeable toe, point (A) will move over a distance of 30 ft. to the right into position (A'), while the lower end (B) of the saturation line will retain its position. Hence, the characteristic dimensions of the dam will be: $h_1' = 98$ ft. and $x_1' = 117$ ft., corresponding to an average hydraulic gradient of 0.84.

Equation (2) represents the equation of the theoretical line of saturation based on the Chézy formula. Graphically, this line is shown by curve (S) in Figure 10. Since, however, the dam is limited on the upstream side by an inclined slope, it is obvious that the saturation line must intersect the slope at a right angle. This conclusion has been repeatedly confirmed by both tests and theory. For perfectly homogeneous fills, it has been found that the line of saturation is a curve with two points of inflection similar in shape to curve (S_1) in Figure 10.

Since the equation of the true line of saturation is known, it is possible to draw line (S_1) accurately. However, this would mean a waste of effort, because slight variations in the coefficient of permeability will cause a considerable difference between the true and theoretical lines, and such variations must be considered inevitable. It is sufficient to know that the true line of saturation will be located somewhere near the straight line A-B.

Since a thorough knowledge of the hydraulic conditions within the dam will be of great assistance in planning the future extension, it was suggested (1) to provide within the dam a series of perforated standpipes for the purpose of determining the true position of the line of saturation and (2) to collect the seepage water within the downstream toe in a drain, equipped with means for measuring the quantity of leakage.

It remained to form an opinion on the possible effect of seepage on the stability of the slopes. Due to the mixed-grained character of the dam-construction material and its high percentage content of angular, coarse-grained constituents, the stability of the dam itself seems to be more than ample. On the northeastern slope there are, underneath the hardpan blanket, layers of rather uniform, permeable reddish sand, through which the hydrostatic pressure may be transmitted to points located below the dam. On this slope the hydrostatic pressure may cause a local rupture of the less permeable blanket which in turn may produce sloughing of the material. During the first year after the dam is placed into use, this slope will have to be carefully watched so that drainage may be provided, if necessary, before important movements occur.

The right slope consists of very stable material. Yet its erratic composition makes it necessary to consider the possibility of subterranean erosion

around the south wing of the dam, along individual streaks or veins of permeable material. In order to avoid the danger resulting from such action, it was suggested to extend the core wall into the slope for a distance at least equal to half the hydrostatic head acting at the same elevation.

Selecting the Dam-Construction Materials.

The information obtained by exploring the proposed borrow-pit area on the left side of the dam was sufficient to show that by far the most suitable dam-construction material is located above elevation 560 (more or less) on top of the deposit of reddish sand. (See Fig. 8.)

By excavating additional test pits we learned that the composition of the deposit in the borrow-pit area is almost as variable as the composition of all the other deposits found in the vicinity of the dam site. Thus the question arose as to how we should determine during construction whether any newly-opened section of the borrow pit would furnish material with a sufficiently low average coefficient of permeability to permit its use. Since by necessity tests are restricted to samples selected at random, while the coefficient of permeability is apt to change from foot to foot, it seemed preferable to obtain approximate results for a great number of samples rather than accurate results for a few.

To reduce the time and labor required for estimating the coefficient of permeability of samples, we developed a method based on the fact that the speed of capillary rise of water through soil becomes less as the soil is less permeable. The theory of the process is somewhat complicated, because the speed of upward movement depends not merely on permeability but also on the ultimate height of the water level. Due to this complication there merely exists an approximate proportionality between the speed of capillary rise and the reciprocal value of the coefficient of permeability. A further complication is introduced by the fact that some air is retained within the voids of the soil while the water moves upward. Nevertheless, the fact remains that the steepness of the curve, which represents graphically the speed of capillary rise, increases with the permeability of the material. Figure 11 shows curves for those samples whose permeability was accurately determined. By observing the capillary upward movement for any sample and by plotting the results on Figure 11, one can roughly estimate the coefficient of permeability of the material.

The device for making the tests is shown in Figure 12. It consists of a simple tank (A) made of sheet tin, an irrigator for feeding the tank, and a set of glass tubes (G), made of hard glass. Hard glass is essential, otherwise the cylinders invariably break during the tests. The bottom of each glass tube is covered with a brass wire screen stretched across and soldered to a brass ring. In order to test a sample, the procedure is as follows:

The sample is dried on a stove, after which it is placed in one of the glass tubes, in layers 2 cm. thick, until the cylinder is completely filled.

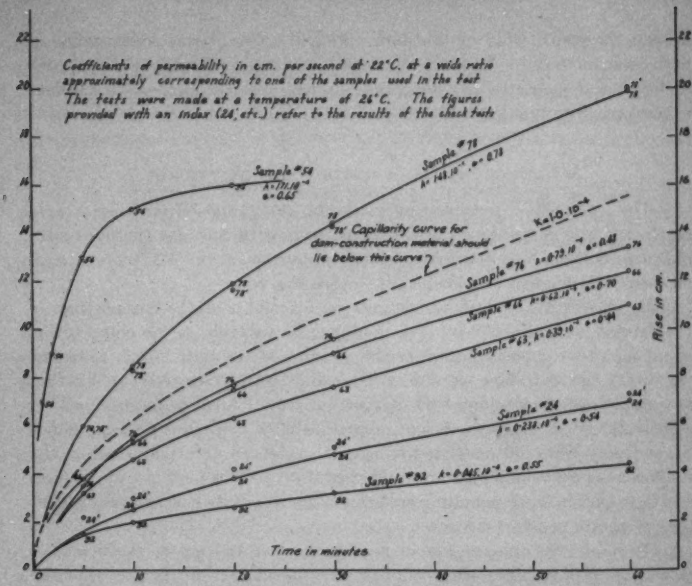
Coefficients of permeability in c.m. per second at 22°C. at a void ratio approximately corresponding to one of the samples used in the test. The tests were made at a temperature of 26°C. The figures provided with an index (24, etc.) refer to the results of the shock tests
Sample #54
Sample #78
Sample #74
Sample #44
Sample #63
Sample #24
Sample #82
"K" Capillarity curve for dam-construction material should lie below this curve
Time in minutes
Rise in cm.

FIG. 11. — CAPILLARY RISE OF WATER IN SOIL SAMPLES.

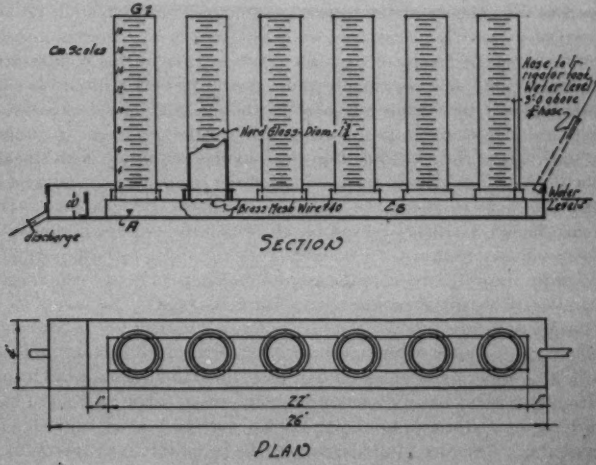

FIG. 12. — DEVICE FOR OBSERVING CAPILLARY RISE OF WATER IN SOIL.

Each layer is compacted by lifting and dropping ten times from an elevation of 4 in. a cylinderical pestle made of hard wood, with a diameter of 1¼ in. and a weight of 150 grams. In case the sand grains are smaller than the openings of the sieve, a 2- or 3-mm. layer of coarser sand is placed on the sieve before the first layer of the sample is introduced. In the meantime the tank is filled with water supplied from the irrigator, the excess water passing over the crest of the overflow (W) and draining away through the discharge pipe. As soon as the cylinder is filled with the sample, the brass sieve is wetted with a moist cloth and the cylinder is placed on the immersed support (S), as shown in Figure 12. Care should be taken to keep the cylinder in a slightly inclined position while lowering it through the surface of the water so as to offer a chance for lateral escape of air.

As soon as the base of the glass cylinder touches the surface of the water, the water starts to rise by capillary action within the sample, as indicated by the color of the sample turning dark. The smaller the permeability of the sample, the slower is the upward movement of moisture. Readings of the rise in water level are taken 5, 10, 20 and 30 minutes after the test is started. The simplicity of the necessary operations permits six tests to be carried on simultaneously.

Observations During Construction.

After the report on the soil studies was presented, the underground strata were exposed to direct inspection by the construction of the core trench and the excavation of the borrow pits. It was found that the underground conditions checked with the predictions. The right (southern) wing of the dam rests against a very feebly permeable portion of the south slope. The soil conditions encountered within the borrow-pit area conform to the cross section shown in Figure 8. The permeability of the material from the borrow pit averages, according to the results of the capillarity tests, 0.000068 cm. per sec., which is below the value of 0.0001 cm. per sec., on which seepage computations were based. The layer of boulders, which covers the bottom of the valley is strictly confined to the area between the slopes. Although the material was found to be very compact, the core wall* was constructed so as to prevent subterranean erosion along individual seams or channels. The bed rock was more badly shattered than was expected from the drilling records; as a consequence it had to be grouted.

In order to exclude borrow-pit material that is more permeable than we intended to tolerate, a curve (k) was traced on Figure 11, corresponding to the capillarity curve of a material with a coefficient of permeability of 0.0001 cm. per sec., on which the estimate of seepage losses was based. From time to time, particularly in case a stratum of doubtful quality was encountered, samples were tested by means of the apparatus shown in Figure 12, with the intention of excluding from use in the dam any material whose capillarity curve is located above curve (k). The material in which the steam shovel worked, was more or less stratified, but when it broke off

*See Figure 4 in the preceding paper of this Journal by E. B. Myott.

the bank and was picked up by the shovel, it became well mixed so that the samples taken from the trucks were fairly representative.

During construction, two questions arose concerning methods for testing embankment material: First of all, whether or not the drying of the material in the stove has any influence on the results of capillarity tests, and second, whether the volume of voids of the material in the embankment is smaller or greater than the volume of voids obtained when compacting the material in the standpipe of the capillarity apparatus. Due to the great

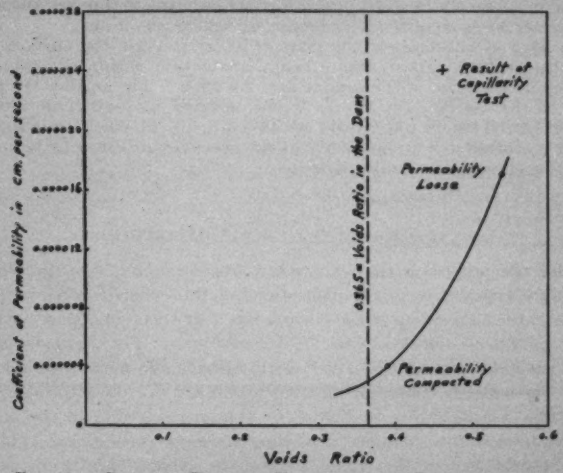

FIG. 13. — RELATION BETWEEN VOIDS-RATIO AND PERMEABILITY OF LOOSE AND COMPACTED SOIL.
Note Result of Capillarity Test.

influence of the volume of voids on permeability, the second question was of vital importance.

Concerning the first question, we found that the drying process had no appreciable effect on permeability, inasmuch as the tests with air-dried and with stove-dried samples gave practically identical results. In order to answer the second question, undisturbed samples were secured from a pit excavated in the finished embankment. Excavation of the pit was difficult and required the use of pick and shovel, which demonstrates the excellent results obtained by compacting with Caterpillar tractors. Due to the presence of many stones in the fill, a cylindrical sampler with cutting edge could not be used. Hence, large chunks of soil (up to $\frac{1}{4}$ cu. ft.) were removed and sealed on the ground by repeatedly painting their surfaces with hot paraffin. In the laboratory the volume of the sealed chunks was determined by immersing them in water. Afterwards the chunks were

skinned and the original and dry weight of the chunks and the specific gravity of the solid constituents were determined. From these data was computed the volume of voids of the more fine-grained mass that filled the interstices between the stones. This volume of voids was found to be equal to 26 per cent. of the total volume. Finally, the more fine-grained material was transferred into a permeameter, and the permeability was determined in a loose and in a compacted state. At this time we learned that it was practically impossible to bring the density of the material in the permeameter, by ramming it in a moist condition, up to the density of the material in the embankment. Figure 13 shows the test results. These indicate that permeability decreases with decreasing voids-ratio much more rapidly than one would expect according to Figure 6. This was due to the appreciable mica-content of the sample. From previous experience we know that the finer the soil and the greater the percentage of scale-like particles, the steeper is the voids-ratio-permeability curve. Figure 13 also shows that the voids-ratio of the sample compacted in the standpipe of the capillarity apparatus is considerably greater than the voids-ratio of the material in the embankment (0.464 against 0.366). Hence, the factor of safety against excessive percolation is ample when computations are based on the results of capillarity tests.

Conclusions.

The soil studies described in this paper secured the following advantages: They made it possible to select the location for the right (southern) wing of the dam in such a manner as to reduce to a minimum the seepage through the underground around this wing. The tests furnished in advance reliable information on the quality of the borrow-pit material and on the boundaries of available deposits. They made it possible to establish specifications for the dam-construction material, based on the properties of the supply available, and to make sure that seepage through the dam will not exceed a certain tolerable quantity provided borrow-pit operations are conducted according to specifications. Finally, the tests indicated the possibility of important seepage losses through the southern slope of the reservoir and permitted consideration of this possibility in the financial plan of the enterprise. The importance of these findings seems to justify the expenditure of the money and labor required to make these studies.

The experiences gained during the investigations suggest the following improvements: The sampling in test pits should be done under the personal supervision of the engineer in charge of the investigation. This would make it possible to concentrate attention from the very beginning on those strata that are of vital importance in the problem, obtain reliable information concerning the location and thickness of these strata and avoid wasting time and labor on others. The method of securing information about permeability by means of mechanical analysis and density determinations

was difficult and very troublesome. We selected this method under the impression of previous discouraging experiences with filter-skin effects. Since then we have learned how to overcome experimental difficulties. Based on our experiences with the Westfield samples, we believe that any further efforts to search for a relation between the coefficient of permeability and the effective size or the specific surface of the material would be time wasted. It is cheaper and more satisfactory to determine permeability by percolation tests. According to our present practice, we make mechanical analyses of only a few representative samples and determine the permeability of others by the direct method, in the densest state we can artificially secure, provided the material in Nature is dense.

The method of keeping track of the quality of the dam-construction material by means of capillarity tests was found to be satisfactory except in so far as the relation between permeability and the speed of capillary rise depends to a certain extent on the character of the sand grains and silt fraction. For this reason two materials with equal permeability are apt to have two somewhat different capillary-rise curves, and a set of curves similar to those in Figure 11 cannot be depended upon except for the same class of materials. In order to eliminate this shortcoming, modifications of the testing procedure are under consideration.

Our opinion concerning the stability of the slopes of the dam was merely based on judgment and previous experience. Since the report was prepared, we have, however, succeeded in constructing a device for measuring those constants (cohesion and internal friction) on which the stability of the slopes depend. By using this device it will be possible to supplement engineering judgment by computations based on reliable test results.

Finally, a few remarks may be appropriate concerning the degree of accuracy of seepage estimates (in our case the estimate of seepage through the dam and through the slopes of the reservoir). The principal sources of error are twofold: (a) Inadequate knowledge of soil conditions beyond the lines explored by means of test pits or of variations within the body of the dam, and (b) differences between the true permeability average and the average of the permeability of samples collected more or less at random. Due to the importance of these two sources of error, it would be useless to attempt more than the computation of the maximum quantity of water that could possibly escape through the soil under the geological conditions found to exist at the site. This means that in making computations, one must introduce for the unknown soil factors the most unfavorable values that can be justified. Thus, when estimating the seepage loss for the south slope of the Granville Reservoir we concluded, from the geological history of the deposit, that the permeability decreases with the distance from the reservoir. However, since the true rate of decrease cannot possibly be ascertained at reasonable expense we assumed the most unfavorable case, viz., uniform permeability. If one knows the upper limit of seepage loss, one is also in a position to know whether or not the necessity for extensive

scaling operations may arise, and one can make the required financial provisions beforehand. However, due to the absence, in Nature, of perfectly uniform soil deposits, no reliable forecast of the actual quantity of seepage losses will ever be possible. Hence, the problem does not consist in computing a definite seepage quantity but in utilizing the available, inadequate evidence and the geological data for estimating the most unfavorable possibilities.

The principal difficulty connected with such estimates is our lack of experience concerning the permeability variations within natural soil deposits of different geological origin. Due to this lack of experience we do not yet possess a reliable basis for estimating how many samples are required for arriving at a satisfactory average. To eliminate this difficulty would require detailed investigation of characteristic exposures and in addition the measurement of the actual seepage losses through slopes whose permeability has previously been estimated. In this connection the measurements that will be made at the Granville Dam promise valuable information.

PROCEEDINGS.

FEBRUARY MEETING.

BOSTON CITY CLUB, BOSTON, MASS.
WEDNESDAY, FEBRUARY 13, 1929.

President Arthur D. Weston in the Chair.

The Secretary announced the results of the letter ballot on the adoption of the resolutions proposed by the Committee on Cross-connections as follows:

Total Number of Ballots cast		341
Yes	304	
No	28	
Irregular	4	
Blanks	2	
Unsigned	3	341

Endorsed by G. H. Finneran and Richard H. Ellis as Committee.

The Secreary announced that at the meeting of the Executive Committee on February 13, 1929, the following had been elected to membership in the Association: *Members:* Richard F. Comerford, Foreman, Water Department, Worcester, Mass.; Ralph G. Lingley, City Engineer, Worcester, Mass.; Joseph E. Conley, Superintendent of Public Works, Norwood, Mass.; Leon A. Goodale, Water Commissioner and Registrar, Worcester, Mass.; Arthur E. Gorman, Wallace & Tiernan Co., Bellville, N. J.; Lester Priest, Clerk, Water Board, Brookfield, Mass.; Walter Knowles, Assistant Engineer, Metropolitan District Water Supply Commission, Enfield, Mass.

Associates: Walter W. Field & Son, Cambridge, Mass.; Mansfield Foundry Co., Mansfield, Mass.

The Secretary read the following communication:

"January 11, 1929.

To the Executive Committee of tbe
New England Water Works Association.

Gentlemen:

The undersigned members of the NEW ENGLAND WATER WORKS ASSOCIATION, hereby nominate for honorary membership, the following gentlemen:

FRANK A. MCINNES, distinguished as an engineer in charge of the water works of the city of Boston; a past president of this Association, to which he has contributed a number of important papers and discussions.

HARRISON P. EDDY, eminent sanitary engineer, holder of the Norman Medal of the American Society of Civil Engineers and of the FitzGerald

Medal of the Boston Society of Civil Engineers; past president of the Boston Society of Civil Engineers.

CALEB M. SAVILLE, distinguished as an engineer and as manager of the Hartford Water Works; author of many papers on water-works subjects and twice awarded the Dexter Brackett Medal of this Association; holder of the Norman Medal of the American Society of Civil Engineers; past president of this Association.

HARRY W. CLARK, noted chemist and biologist; for many years in charge of the laboratories of the Massachusetts State Department of Public Health; whose investigations have led to many of the most notable advances in treatment of water and sewage; author of many papers presented to this Association and holder of its Dexter Brackett Medal.

Respectfully submitted,

(Signed) CHARLES W. SHERMAN
FRANK A. BARBOUR
RICHARD H. ELLIS
THOMAS E. LALLY
DAVID A. HEFFERNAN
SAMUEL E. KILLAM
ROGER W. ESTY
FRANK E. WINSOR
GORDON M. FAIR
ARTHUR E. BLACKMER."

THE SECRETARY. At the meeting of the Executive Committee on February 13, 1929, these gentlemen were unanimously elected honorary members of the Association. (*Applause.*)

THE PRESIDENT. The following have been nominated as a Committee on the Award of the Dexter Brackett Memorial Medal for this year: Gordon M. Fair, Chairman; Frank A. McInnes; G. Earl Lourie.

We have also named the following as a Committee in charge of the Annual Convention to be held in Portland on September 17, 18, 19 and 20, with headquarters at the Eastland Hotel. The Committee is: James W. Graham, Vernon F. West, Hon. William F. Sullivan, Mayor of Nashua, N. H.

The Executive Committee this morning consented to the President's calling a special meeting of the Association to be held on April 10 at Waterbury, Conn., at the Hotel Elton. The following will be the members of the committee for carrying out that meeting: Theodore F. Bristol, of Ansonia, Conn.; J. Warren Scott, of the Bureau of Sanitary Engineering, State of Connecticut; Mr. Kennedy, Superintendent of Public Works at Waterbury, Conn.; G. Earl Lourie, Superintendent of the Water Department at Bristol, Conn.; R. A. Cairns, City Engineer, Waterbury, Conn. Mr. Lourie will be the Chairman of the Committee.

At this morning's meeting of the Executive Committee the question of closer supervision over legislative matters was brought up, and the Executive Committee decided to appoint a steering committee on legislative matters. There will be one member on that committee from each of the New England States, with a chairman from Massachusetts, the idea being that when any matters affecting water-works practice in any of these states is about to be presented, or is being presented in any of the various Legislatures or General Court bodies, it will be called to the attention of this

Committee, and they will meet and appoint a spokesman to appear at Committee meetings of the Legislature.

A paper on "The Granville Dam at Westfield, Mass.," was read by C. A. Farwell, Partner, Fay, Spofford & Thorndike, Consulting Engineers, Boston, Mass.

A paper on "Construction Features of the Granville Dam," was read by E. B. Myott, Resident Engineer, Fay, Spofford & Thorndike, Consulting Engineers, Boston, Mass.

Dr. Charles Terzaghi, Associate Professor of Foundation Engineering, Massachusetts Institute of Technology, discussed the soil mechanics and geology of the Granville Dam project.

The papers and discussion were illustrated by stereopticon views.

On motion of Stephen H. Taylor, duly seconded, a rising vote of thanks was extended to the speakers.

(*Adjourned.*)

March Meeting.

Boston City Club, Boston, Mass.
Wednesday, March 13, 1929.

Past Presidents' Day.

President Arthur D. Weston in the Chair.

The Secretary announced that at the Executive Committee Meeting of March 13, 1929, the following had been elected to membership in the Association: *Members:* Roger W. Macdonald, City Engineer, Peabody, Mass.; Harry B. Collins, Superintendent of Public Works, Reading, Mass.; Arthur V. Harrington, Assistant Engineer, Massachusetts Department of Public Health, Stoneham, Mass.

Associates: The Bond Co., Boston (Chester H. Tucker), Bennett Company, Somerville (D. C. Bennett).

The President. When your Budget Committee reported this year they made no provision for assisting in financing the committee work of the Standards Committee. In the past we have contributed, along with the American Water Works Association and some other associations and the producers. In 1924 we voted to appropriate $400 for that work. We have not been called upon since that time, but we are now called upon. We cannot legally appropriate the money without the consent of the Association at this meeting, and I am going to call on Mr. Killam to explain the need of the appropriation.

Samuel E. Killam. On December 9, 1924, it was voted by the New England Water Works Association to participate in the sponsorship of the Cast-iron Pipe Committee as outlined by the American Engineering Standards Committee, the name of which has now been changed to the American Standards Association. The Sectional Committee was spon-

sored by the following societies: American Gas Association, American Society for Testing Materials, American Water Works Association, and the NEW ENGLAND WATER WORKS ASSOCIATION, all of which contributed $400 each to the general fund. This contribution was in 1927. Before that considerable money had been expended for clerical work but from the general fund. There has been made available for use of the Sectional Committee over $60 000. Most of this money had been contributed by the producers. On November 30, 1928, there was in the hands of the Treasurer $39 000 from the original fund. The Sectional Comittee has asked the four sponsor bodies to meet the clerical and stenographic expenses, which include one-half the salary of the executive assistant to the Chairman. The four sponsor bodies have been asked for the following amounts: $1 000 each from the American Gas Association and the American Water Works Association, and $400 each from the American Society for Testing Materials and the NEW ENGLAND WATER WORKS ASSOCIATION. A progress report of the work which has been done has been prepared and sent to the sponsor bodies. This report gives an idea of the importance of the work under way. The money is being spent in large degree for research at the Massachusetts Institute of Technology, Columbia University, the University of Illinois, and the University of Iowa. Those of us who are on the various committees appreciate the magnitude of the work. I had been in hopes that the Committee that I represent would be able to make its final report March 26, but I understand that more work has been added to be done. I personally represent the American Water Works Association.

I move, Mr. President, that the sum of $400 be appropriated for this work.

(*The motion is duly seconded and carried.*)

THE PRESIDENT. As you all know this is Past Presidents' Day. The reason for this is twofold: In the first place, a great many of the younger members of the Association are not acquainted with the Past Presidents. We think that it is a good thing for the younger members to know the Past Presidents of the Association.

The second reason is that a good many of the Past Presidents do not have an opportunity to come to our meetings. We thought that this would be an incentive for some of them to come. They have responded handsomely. Out of 22 Past Presidents of the Association, 19 are here today.

Secretary Gifford then introduced the following Past Presidents who were greeted with hearty applause: Byron I. Cook of Woonsocket, R. I.; Stephen H. Taylor of New Bedford, Mass.; Frank A. Barbour of Boston, Mass.; Charles W. Sherman of Boston, Mass.; J. Waldo Smith of New York, N. Y.; Allen Hazen of New York, N. Y.; Frank E. Winsor of Boston, Mass.; Percy A. Sanders of Concord, N. H.; Caleb Mills Saville of Hartford, Conn.; Theodore L. Bristol of Ansonia, Conn.; John C. Whitney of Newton, Mass.; Frank A. McInnes of Boston, Mass.; David A. Heffernan of Milton, Mass.; Robert J. Thomas of Lowell, Mass.; Fayette F. Forbes

of Brookline, Mass.; Frank E. Merrill of Somerville, Mass.; George A. King of Taunton, Mass.; Hon. William F. Sullivan of Nashua, N. H.; Samuel E. Killam of Boston, Mass.

THE PRESIDENT. Some of our honorary members are also here, some of whom are Past Presidents of the Association. Mr. X. H. Goodnough, of the State Department of Health, was introduced.

We had hoped to have the Governor with us today. He could not come, but he has sent a very worthy substitute, — the Honorable Frederick W. Cook.

REMARKS BY HON. FREDERICK W. COOK.

Mr. President, and members of the NEW ENGLAND WATER WORKS ASSOCIATION:

The Governor has sent this letter to your President:

"My dear Mr. Weston:

It is a matter of sincere regret to me that I am unable to accept your kind invitation to attend the meeting of the NEW ENGLAND WATER WORKS ASSOCIATION to be held at the City Club today. Until the last day or so I had hoped that I might have the privilege of being present and of extending on behalf of the Commonwealth a welcome to the members. I would have been especially glad to have been there to present the certificates of honorary membership to those who have contributed so much to the Water Works practice in the New England States. Their service indeed merits special recognition and consideration.

With cordial greetings and every good wish, believe me

Very sincerely yours,

(Signed) Frank G. Allen."

I am really sorry for you, and also for the Commonwealth, that the Governor could not come here today. I would really like to have had him have the privilege — and it is a better one for him to have than for me, because he is the chief executive, and he is dealing with the executive work of the Commonwealth, and that includes cities and towns, all the time — I would like to have had him have the opportunity of seeing you men and meeting the men who are doing so much for our Commonwealth.

This whole matter of the water works in our cities and towns is extremely interesting to all of us who study government. Every time a proposal is made to have the State take over some particular public utility, some particular industry that is today carried on by private enterprise, and the claim is made that it cannot be carried on under governmental supervision with economy or with benefit to the people, the water works is cited at once as an example of what can be done. And it certainly is a very convincing argument. If we could be assured that our other utilities would be carried on as the water works have been, it might be just the last thing needed to put these different measures across. But we are not assured that this would be so.

I have been impressed today as I have looked at the men throughout the audience, many of whom are men that I know, men who have been in my office to discuss different matters of town business, and these other men, probably the most outstanding engineers of the United States, and I realize that the reason for the success of water-works management is that you get together and work out these problems for the common good.

We often hear criticism of our motor-vehicle department. We hear criticism of appropriations, we hear talk that the Commonwealth of Massachusetts uses for other than road purposes money received from the licensing fees for automobile registration and operation. I remember years ago, when I was City Clerk of the city of Somerville, when Frank Merrill was Water Commissioner, as he is today, that year after year we would put in our budget of estimated receipts a very large amount for us, as a city of 100 000 people, to be applied to water-loan bonds and water-loan interest, and we always still had remaining a great many dollars that we applied to other purposes. I think that this is a parallel case and yet we hear criticism. On the other hand, the rates for water in our cities and towns have been kept down, — way, way down.

I know my Dad bought a place up country a few years ago, where they had no water supply. There was a little hill up back with a spring on it. I think he spent something like $400 or $500 to pipe the water down to his house. When he showed me the bill I was astounded, because Frank Merrill was selling me water at about $3 a quarter. Dad was paying overhead enough there to keep about ten houses supplied with water in the city. You see that you have done something as you worked together, and it has been, not for just your own selfish purposes, but it has been for the common good of all the cities and towns of Massachusetts and New England.

I shall not take any more of your time. The Governor sent me to extend his greetings to you, and really it is a very great pleasure indeed to do so, — to you, Mr. President, and to the members of the Association. (*Applause.*)

THE PRESIDENT. At the last meeting of the Association you were told of the action of the Executive Committee regarding certain nominations for honorary membership, and at that time the following were elected to honorary membership: Harry W. Clark, Director and Chief Chemist, Mass. Dept. of Public Health, Boston, Mass.

Harrison P. Eddy, of Metcalf & Eddy, Consulting Engineers, Boston, Mass.

Frank A. McInnes, Consulting Engineer, 6 Beacon Street, Boston, formerly connected with the Water Department of the city of Boston.

Caleb M. Saville, Manager and Chief Engineer, Board of Water Commissioners, Hartford, Conn. I shall ask Mr. Cook to present the certificates of honorary membership to these men.

(*The certificates are presented by Mr. Cook.*)

HARRY W. CLARK. Mr. President and members of the Association. It is very pleasant to receive this certificate of honorary membership in this Association. Yet I have noticed on occasions like this that it is usually the older men who get them, and so it makes us all realize that we are on the shady side of life.

We four men who are to receive these certificates today may feel like the revolutionary soldiers at Bunker Hill when Webster turned to them and said, "Venerable men who have come down from former generations." That is about the way I feel here, because I have been a member of this Association thirty-three or thirty-four years, and a very large number of the men who were members then have passed away. It is a very different set of faces today from those I looked into thirty years ago. I was about thirty years old when I joined the Association, and I think that these men, some of them, who come here today, look about that age. I thank you very much. (*Applause.*)

FRANK A. MCINNES. Mr. President, the NEW ENGLAND WATER WORKS ASSOCIATION has always been very, very dear to me, and for that reason I can honestly and sincerely thank you for the honor you have done me. Father Time has been lenient so far with me, and I only hope he will continue to be so that I shall be able to show by my actions that I appreciate the honor you have given me. (*Applause.*)

HARRISON P. EDDY. Mr. President and fellow members of the NEW ENGLAND WATER WORKS ASSOCIATION: This Association has stood for advancement in the water-works field for a great many years, being one of the oldest associations of its kind in the country.

There is no greater honor and no greater source of satisfaction that can come to a person than to be recognized by his contemporaries and by the men with whom he comes frequently in contact. If the men who know him so well, think highly enough of him to pay him honor, it certainly should be a source of great satisfaction. And I sincerely appreciate the honor that you have done me today in this election to honorary membership. (*Applause.*)

CALEB M. SAVILLE. I appreciate this honor that you have conferred on me today more than most anything else that could come to me in this line. I have been a member of this Association, I guess, pretty nearly as long as, if not longer than any of those here. I am more glad to receive this honor on account of the intimacy, perhaps, that it brings to me with the members, the great members of the water-works engineering profession who have now passed on, who made this Association what it is, not only locally in New England but all over the world, because I think there is no other water-works association that has had a more outstanding place in water-works science than the NEW ENGLAND WATER WORKS ASSOCIATION.

Gentlemen, I thank you very sincerely for the honor that you have given to me. (*Applause.*)

THE PRESIDENT. During the year 1928 there was an amendment to

the General Laws which has affected seriously the finances of water departments and municipal departments throughout the State. There has been considerable confusion in the minds of a great many of our members as to just what is the meaning of that amendment. Mr. Waddell, who is Director of the Division of Municipal Finance in the State, is here today, and he has consented to speak a few words to you. Mr. Waddell. (*Applause.*)

REMARKS BY MR. THEODORE M. WADDELL.

Mr. President and Gentlemen of the NEW ENGLAND WATER WORKS ASSOCIATION: The Act of 1928 did not, in my opinion, seriously interfere with the management of water works. I know that most of you feel that to be the case. You appreciate, of course, that the Act of 1928 was not for the water works alone; it was for all departments of a municipality and it was, in brief, carrying out the municipal finance laws which required an appropriation for every department.

The question arises, what shall be done with the water revenue? Shall it be used to reduce taxation, or shall it be maintained and held for the water department alone? This is an opportunity which I welcome, to set clear in the minds of many of you the attitude of our Department.

We have been constant advocates of appropriations for each and every department of the municipality. We firmly believe that no official board is greater than the community itself. I think if you will turn back to the meeting of 1924, you will find at that time we advocated the holding of water revenue for water purposes alone. We advocated at that time the raising by taxation of a reasonable sum for hydrant service. We did not oppose the payment of a reasonable tax so as to make the water supply an honest-to-goodness enterprise. What we advocated then we advocate today. Since that meeting I have had hundreds of conferences with different officials, and in every case I have tried to make our stand clear, that the water revenue should be held for water purposes, that water rates should be raised or lowered as the case may be so that the taker would be called upon to contribute only what was necessary to maintain a reasonably safe and sane water supply system.

I think what your President wants, however, is to know the stand taken by the recent Commission which was to report on municipal expenditures. That report has been filed, but unfortunately is not in print. While the Commission did not report a bill, they did report a recommendation that water revenue should be held for water supply purposes. They have recommended that the water rates should be such as will properly maintain the system. They have recommended that no portion of the water rates be used to reduce taxation, other than possibly the amount that would be contributed by way of taxation were it privately owned.

To my mind their recommendations are all right. To my mind this Association should see to it that a bill carrying out those principles, if those

principles meet with your approval, be filed when this Commission report is to be considered, or is considered by the Municipal Finance Committee, which undoubtedly will be the case before the session closes.

I know of no real, honest objection to the recommendations of that Committee. There are a lot of objections to using water revenue to reduce general taxation. Wherever we find that to be the case, I fear that it is sure ruination or wrecking of the plant. A good many times officers are elected, both as mayors and as councilmen, on the strength of what they have done to reduce the tax rate.

In 1928 our department made a report somewhat along the same lines, and while we made no attempt to introduce bills which would carry out all those recommendations, we introduced some three or four bills which were subsequently enacted into law. This year the Recess Commission have introduced approximately six or seven bills. I was not anxious to have the Committee introduce 17 bills, because you would have a report so big that all would throw up their hands the moment they saw it in printed form.

There is just a little psychology in legislation from a layman's point of view. Of course the Secretary here can tell you more about how to handle the political side of it than I can, because I am not supposed to know anything about the politics; in fact, I know that I do not. However, I have seen the workings of committees in the last few years, and I have not the slightest doubt that legislation, proper legislation, can be enacted during the present session. I am more than anxious that something be done to ensure that water revenue be used for water purposes, and I would like to make it impossible for municipalities to borrow for water purposes and at the same time use money that they have properly collected for water service to reduce taxation.

Unfortunately, many of our towns have failed to recognize the 1928 Act. I have it put to me almost scores of times each and every day that they knew nothing about it. I also happen to know that House Bill 192 was heard last year three different days, the room was full, and they saw one little item where we had recommended that for borrowing inside the debt limit there should be a contribution of 50 cents instead of 25 cents, as found in the old law, and they spent the entire time arguing on that one point and overlooked everything else that was in the report. How on earth you are going to make people read the entire report is beyond me.

I have another bill in mind, by way of illustration, on which they stated, "We knew nothing about any legislation contemplated this year," and I know personally that 2 000 letters were sent out, broadcast throughout this Commonwealth, asking for comments, recommendations, opinion as to whether this bill should be enacted. And yet they know nothing about it.

Now, it is not our fault. We do not want to put anything over. We want to do the right thing. But understand, gentlemen, that while I am interested in the water works, I am interested in every department in the municipality, and we are trying in our weak way to make recommendations

along the line of principle. I do not believe it is fair to allow a community to wreck its system of water supply merely to make a good showing in taxation.

I have had some experience there, especially when two Boards will come in and love one another to such an extent that they will not ride in the same car. It makes an interesting hearing. Yet in this particular case I am convinced in my mind, — although I confess my ignorance on water construction, — that they have not put enough in the plant, that they have accumulated a surplus improperly and at the expense of the plant.

Only yesterday my attention was called to an accumulation of a sum equal to 40 or 50 per cent. of the entire year's revenue as a surplus. It should not be so. If you have a rate which creates a surplus of 40 or 50 per cent. of the year's revenue, your rate is too high, and it should come down. If you have a rate that does not give adequate funds to cover your annually recurring charges for laying and relaying the mains, the rate should go up. I think it is a fair proposition. I do not want to divorce the idea of an enterprise from the water system, but I do believe that the Lincoln case, in which they decided that all departments of a town make up a municipal entity and that no one department can be a debtor to another, is sound. (*Applause.*)

JOHN F. SULLIVAN.* If a balance is left in the water-department appropriation at the end of the year can the water department use the balance providing it is appropriated by the Council or the Board of Aldermen?

MR. WADDELL. Absolutely.

STEPHEN H. TAYLOR.† If all the receipts of the water department are turned in to the general fund, is there any way by which we can be assured that the receipts will be returned to the water works for its use? It seems to me that, unless it is so returned, the arrangement is very unfair. We might turn over, say, $400 000 in water receipts, and the City Council might be hard up for money for some purpose and say, "We will only give you back $300 000 and you can take the rest out of your plant in some way, by not keeping it up to date." It seems to me that there should be some assurance that the water department will get back the funds that are received for water.

MR. WADDELL. There is some doubt at the present time whether our special water acts will hold that revenue for water supply. The whole object of the recommendation of this special Committee is to remove the doubt. I have on my desk an opinion from a very able firm of lawyers which claims that the Act of 1928 only applies to the appropriation; this follows the recommendations that our office has made from the inception of the Act. However, I would feel very much better, and it would eliminate a great many arguments, if we can make it clear by statute, which I hope will be

*Superintendent of Water Works, Chicopee, Mass.
†Superintendent of Water Works, New Bedford, Mass.

introduced and passed as a result of the report of this Committee during the present session.

ROGER W. ESTY.* Do I understand that under the present law of 1928 a department surplus can be appropriated for another department? For example, if we have a $10 000 surplus can the town or city appropriate it to other departments?

MR. WADDELL. I hope not. There is just enough doubt about it so that I think we ought to make it definite and sure. In the accounting we are setting up a water revenue account, carrying it as a separate revenue account, as distinguished from general revenue, but there is enough doubt that legislation is warranted to make it definite and sure.

ROGER W. ESTY. I think that this Association should recommend to Mr. Waddell to draft the bill and present it to the General Court so that all funds obtained from water revenue be devoted to the water department.

BENTON W. VAUGHAN. If in making up the income revenue the water department income revenue exceeds the estimate by $10 000 or $15 000 can that money be appropriated to the credit of the water department for the whole amount, if the whole amount does not represent the full estimate for all departments?

MR. WADDELL. I am not sure I am clear on that point. If there is a deficit, the deficit has got to be made up, and according to our method of accounting, we could hold that as a deficit in the water department. Here, however, your appropriation will be made, to be taken from water revenue, and held separate from other departmental revenue. But I must again repeat that we would feel very much better if we could point to chapter so and so of the Acts of 1929 which says that that must be done.

BENTON W. VAUGHAN. In a case I am interested in the total income revenue had not reached the estimate in November but the water department had exceeded the estimate by over $10 000, and we were obliged to have a $4 500 overdraft, which is against the law, because there was no money to pay it.

MR. WADDELL. Did they use any of the water revenue to reduce taxation?

BENTON W. VAUGHAN. They took $195 000, the previous year's income, and appropriated back about $40 000 for construction of mains.

MR. WADDELL. How much did they borrow?

BENTON W. VAUGHAN. We borrowed from $25 000 to $50 000 every year and now have a debt of about $125 000. We have made up as high as $110 000 in one year.

MR. WADDELL. It is just those municipalities that have rates either too low or use their water revenue to reduce taxation and borrow outside the debt limit for water mains and water extensions that are annually recurring which ought to be affected by the Act introduced and recommended this year.

THE PRESIDENT. The officers of the Association will follow the sug-

*Superintendent of Water Works, Danvers, Mass.

gestion of Mr. Esty and work along with Mr. Waddell on the matter of additional legislation.

CHARLES W. SHERMAN.* Before you introduce the speaker, Mr. President, I want to take it upon myself, not to offer a motion, but in the name of the members present, whom I am sure I represent, express our appreciation of the arrangements for this meeting. I am sure that the President has done himself proud in bringing out one of the finest meetings the Association has ever had. (*Applause.*)

A paper on "East Providence Water Company Rate Case," was read by Albert C. Dickerman, former Superintendent, East Providence Water Company. The paper was illustrated with stereopticon views. Messrs. Arthur D. Weston, William F. Sullivan, James V. Turner, John P. Wentworth and Henry T. Gidley took part in the discussion.

On motion of Samuel A. Agnew, seconded by Frank E. Winsor, it was VOTED that the following be appointed as a Nominating Committee to bring in a list of officers for the coming year: Stephen H. Taylor of New Bedford, Theodore L. Bristol of Ansonia, Conn., Timothy W. Good of Cambridge, Edward G. Hughes of Watertown, Joseph E. Conley of Norwood.

(*Adjourned.*)

SPECIAL APRIL MEETING.

HOTEL ELTON, WATERBURY, CONN.
WEDNESDAY, APRIL 10, 1929.

President Arthur D. Weston in the Chair.

THE PRESIDENT. First of all I want to thank all the Water Department officials of Connecticut for their part in making this meeting such a success. I am glad to have them here with us, and I am glad to see so many new faces. I hope that next fall we will see many of them in Boston.

I want also to thank the members of the local committee, of which Mr. Lourie was Chairman.

The President announced the election of Harold W. Griswold to membership in the NEW ENGLAND WATER WORKS ASSOCIATION.

A paper on "The Shepaug Tunnel," was read by R. A. Cairns, City Engineer, Waterbury, Conn.

A paper on "Maintenance of Small Meters," was read by H. W. Griswold, Designing Engineer, Board of Water Commissioners, Hartford, Conn.

A paper on "Maintenance and Testing of Large Meters," was read by Roger W. Esty, Superintendent of Water Works, Danvers, Mass. Mr. H. C. Crowell asked a question in regard to the paper.

A paper on "Installation and Maintenance of Venturi Meters," was read by A. B. Coulters of the Builders Iron Foundry, Providence, R. I. Mr. Charles G. Richardson of the Builders Iron Foundry submitted a written discussion of Mr. Coulters' paper.

(*Adjourned.*)

*Of Metcalf & Eddy, Consulting Engineers, Boston, Mass.

GEORGE WELLINGTON BATCHELDER — A MEMOIR.

Died December 6, 1928.

George Wellington Batchelder, son of George E. and Olive M. (Boardman) Batchelder, was born in Georgetown, Mass., March 17, 1862. He became a resident of Worcester, Mass., at an early age and received his education in the public schools of that city. He was elected Water Registrar of the Worcester Water Department in 1899, succeeding his father, George Batchelder, who had held the office since 1886. In 1904, he was elected to the combined office of Water Commissioner and Water Registrar and continued in this position until his death. In his long administration of this important trust he displayed ability, integrity and devotion to duty in a marked degree. The city of Worcester is fortunate indeed in having been served for so long a period by such a man.

Mr. Batchelder was widely known to and highly respected by waterworks men throughout the country. He became a member of the New England Water Works Association in 1899, served as its president in 1912 and was ever deeply interested in the welfare of the Association. As late as September, 1928, at the annual convention in Montreal, he acted as chairman of the committee on golf, a game to which he was devoted and in which he was proficient. His cheery smile and kindly words will be sadly missed by our members.

Mr. Batchelder was also a prominent member of the American Water Works Association, serving as Vice-President in 1925–6; in the following year he declined the nomination for president owing to pressing responsibilities in connection with his work in Worcester. He was also a member of Quinsigamond Lodge A. F. & A. M.

In 1890 Mr. Batchelder married Sara Hathaway by whom he is survived together with a son, Robert E. Batchelder, Lieutenant, U. S. Navy.

Mr. Batchelder's personality was a lovable one, endearing him to all his friends. He was ready at all times to help his fellow man and made the lives of those who were fortunate enough to know him brighter and better.

FRANK A. McINNES,
DAVID A. HEFFERNAN,
STEPHEN H. TAYLOR.

FRED B. FORBES. — A MEMOIR.

Fred Bettinson Forbes, son of Andrew Jackson Forbes and Ella Bettinson Forbes, was born in Cambridge, Mass., on January 30, 1872, and died in Somerville on January 26, 1929. He belonged to two old New England families and was a member of the Massachusetts Society of the Sons of the American Revolution. He attended the Cambridge public schools and was graduated from the Massachusetts Institute of Technology in 1893. He began his life work in July of that year as an assistant chemist of the State Board of Health of Massachusetts at the Lawrence Experiment Station. In 1897 he was made first assistant in the State House laboratories. From that date until his death Mr. Forbes devoted himself unselfishly and faithfully to the work carried on in that laboratory and so systematized it that probably no other laboratory of a like nature in this country turned out so much good analytical work at so low a cost. His particular forte was in overseeing his force and so arranging the work as to make the laboratory most efficient. He had little liking for research, preferring what might be called the business of laboratory operation, although considerable valuable work was done by him from time to time upon improving methods of analysis used in sanitary laboratories. Two or three papers on such subjects were written by him and published in chemical journals. In everything he did he was direct and concise, and he wasted neither time nor words. His mind was clear, and he knew how to express himself in good, clear, terse English. He could thus give information in regard to chemical questions and problems in plain, simple language easily understood by all. Throughout his entire life he never seemed to think of himself but of the work in hand and its value to the Department of Public Health and to the State. As recorded in the resolutions to his memory passed by the State Health Council and published in the "Commonhealth" of January, 1929, "his service has throughout been marked by those qualities found only in a person of sound training, clear thinking, loyalty, conscientiousness and profound self-effacement."

Mr. Forbes was a man of great integrity, and in the opinion of those who knew him and his work most intimately, no other chemist in the country could have carried on this work on a higher plane or with greater efficiency. The State of Massachusetts is fortunate to have had the services of so exceptional a man for so many years. Besides his widow, who was Anna Maud Soule of Cambridge, and two sons, he is survived by his mother and three sisters, all of Somerville. Mr. Forbes became a member of the New England Water Works Association in 1899.

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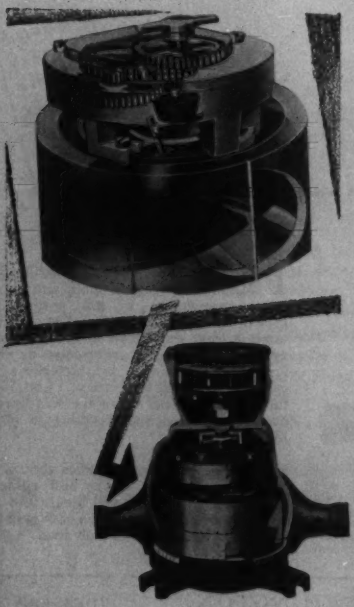

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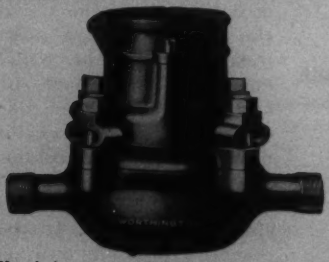

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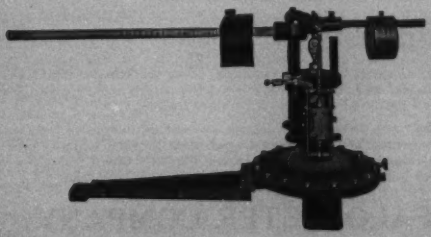

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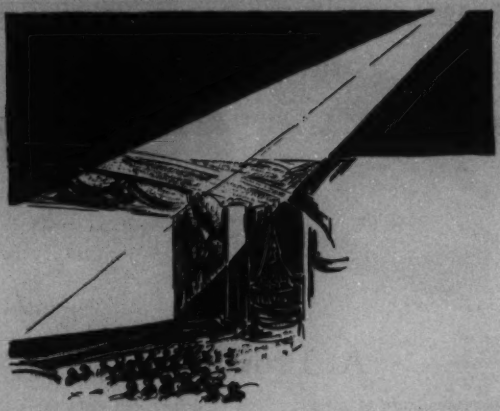

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In 1893 the Kinetoscope (the first practical moving picture projector) was invented by Thomas A. Edison.

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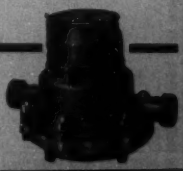
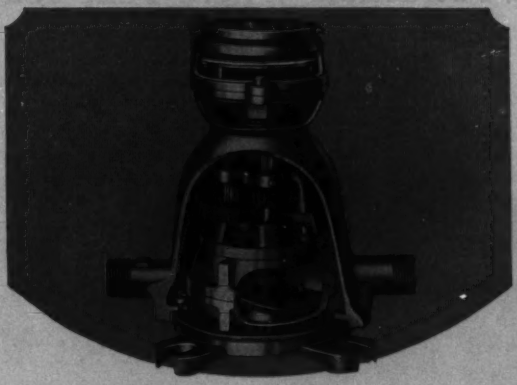


Announcing Our
Sixtieth Anniversary Meter



King F. P.

King F. P. (frost protected) Meter is built to overcome every known weakness in frost meter design. Among the new features built into the King F. P., which users of meters will immediately recognize as marked advances, are the position of the thrust roller, an eccentric which locks the disc chamber in place, and the use of monel metal pins and lock screws which yield to excessive pressure such as frost.

The thrust roller is located on the outlet side of the disc slot, and is held in place by a monel metal pin which balances the weight perfectly and strengthens this edge of the disc, always heretofore the weakest point. Friction thrust is reduced to a minimum because the roller keeps the control in the center. The roller is made of hard rubber and this perfect insulation eliminates corrosion and galvanic action.

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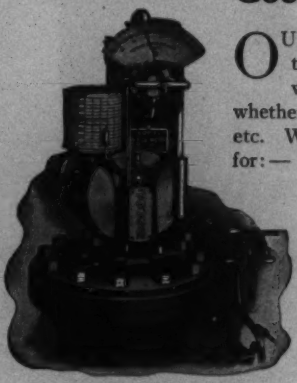

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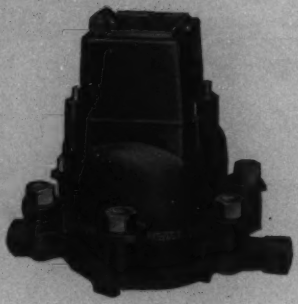

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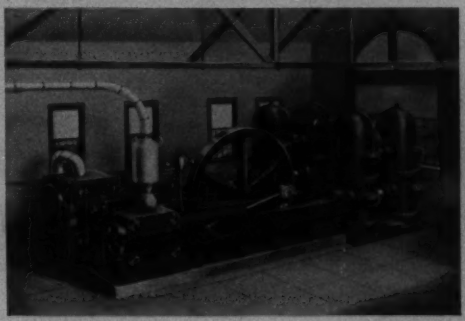

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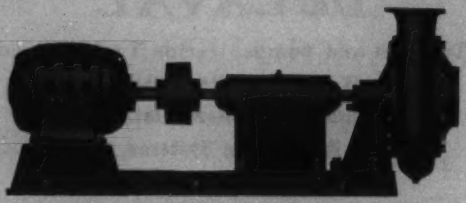

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For the first time a pump has been designed that will handle fluids containing the fibrous and stringy matter found in sewage — without clogging, without preliminary screening, and without the excessive power consumption that characterizes the less satisfactory open impeller type of pump.

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This pump was designed by Mr. A. B. Wood who has been serving the city of New Orleans and other municipalities in the capacity of consulting engineer. It was originated by Mr. Wood to cope with the very unusual sewage problems found in New Orleans and met with immediate success.

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28 branches in principal cities at your service.

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PUMPS · DIESEL ENGINES · MOTORS

DE LAVAL

Steam Turbines and Pumps, Marine Turbines and Gears
Worm Gear Reductions
Oil Purifiers, Clarifiers, Dehydrators
Reclaiming Systems

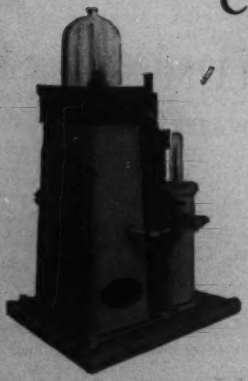
Bath Street Station, Providence, R. I.

New pumping station equipped with two De Laval automatic motor driven and two De Laval gasoline engine driven emergency pumping units each designed to deliver 2 000 gallons per minute of water against 98 feet total head. Contract included furnishing and erection of pumps, motors, gasoline engines, switchboard, gages, tanks and piping.

TURBINE EQUIPMENT CO. of NEW ENGLAND
ENGINEERS AND CONTRACTORS
80 Federal St., Boston **Telephone, Liberty 5995**

New England Representative for
DE LAVAL STEAM TURBINE COMPANY
Trenton, N. J.

Solution Feed CHLORINATORS



30% MORE EFFICIENT *at* New Haven

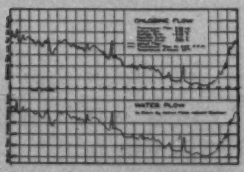


Diagram showing accuracy of W&T Automatic Vacuum Type AS V Solution Feed Chlorinator at Maltby No. 1, New Haven Water Company.

Note: The flow of chlorine varies exactly with the flow of water.

NEW HAVEN—with its enviable record for safe water and freedom from typhoid fever—has just announced a finding of first importance to water works and health officials. *(See page 477 Water Works Engineering, Vol. 82, No. 8.)*

Based on several years experience with chlorination by both solution and direct feed methods of application, they find:

SOLUTION FEED { 37.2% more efficient at Lake Dawson
27.6% more efficient at Maltby No. 1

The higher efficiency of *solution feed* chlorination over direct feed chlorination as proven at New Haven holds wherever chlorine is used—whether for water sterilization, sewage disinfection, swimming pool sanitation, odor control, or for industrial uses.

In many plants the savings in chlorine alone—to say nothing of the maintenance savings and ease of operation—will pay for a duplicate chlorinator.

And W & T *Solution Feed* Chlorinators make use of the vacuum principle of control—that insures long life and rugged dependability.

WRITE FOR TECHNICAL PUBLICATION NO. 99

"The only safe water is a sterilized water"

WALLACE & TIERNAN

COMPANY, INCORPORATED

Manufacturers of Chlorine Control Apparatus

NEWARK NEW JERSEY

NEW YORK CHICAGO KNOXVILLE SAN FRANCISCO MINNEAPOLIS PITTSBURGH DALLAS KANSAS CITY LOS ANGELES SEATTLE ST. LOUIS BUFFALO INDIANAPOLIS DETROIT BOSTON JACKSONVILLE CHATTANOOGA CHARLOTTE OKLAHOMA CITY PHILADELPHIA SPOKANE WALLACE & TIERNAN, LTD., TORONTO, CANADA OGDEN
WALLACE & TIERNAN, LTD., LONDON, ENGLAND

SA-18

CMC

FILTER ALUM

FOR NEW ENGLAND WATER WORKS

MERRIMAC CHEMICAL COMPANY

148 STATE ST. BOSTON

MANUFACTURERS OF FILTER ALUM
SINCE 1886

THE "HACKENSACK"
All-Purpose Tapping Sleeve

OUR "Hackensack" Tapping Sleeve may be readily used for tapping both sides of a pipe line at the same point by employing two branch-connection halves, as shown above, instead of one, as is usually done. "Hackensack" Sleeves are made extra heavy throughout. Lead for the joints is only needed at the ends of the sleeve.

SMITH
WATER WORKS SPECIALTIES & SUPPLIES
A.P.S. 1928

THE A.P.SMITH MFG. CO. *East Orange, N.J.*

The ORIGINAL METER BOX

CLARK

H. W. CLARK COMPANY
MATTOON, ILL., U. S. A.

PRECALKED JOINT PIPE

CAST IRON PIPE

Bell and Spigot—Sand Cast—Modern Weights

Sizes: 1¼ through 12 inches. Lengths to 16 feet

Boston stocks. Furnished with or without Precalked Joints

McWane Cast Iron Pipe Company

Eastern Office:
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Sales Representatives:
FRED A. HOUDLETTE & SON
40 Central Street, Boston
Eastern Mass., Me., N. H.

General Offices: BIRMINGHAM, ALA.
Foundries: BIRMINGHAM, ALA., PROVO, UTAH

Main's Hotcoating for the inside of standpipes and water tanks, will stop all pitting and corroding, and is guaranteed for seven years.

Also

Main's Coverall Salt Air Paint, for the outside of standpipes and water tanks, guaranteed for four years.

M. B. MAIN **Box 189, Somers Point, N. J.**

GRIP PIPE FITTINGS

Connect without threading

As an Emergency Pipe Clamp

Before

After

Makes Possible an Immediate Repair. Saves Both Time and Money

MANUFACTURED BY

WARE COUPLING & NIPPLE CO.

WARE, MASS., U. S. A.

Four Generations Have Passed

since the Croton Aqueduct was built . . . .

Yet the wrought iron pipe is still good for generations more!

Away back in the 60's, before the Civil War, a 90-inch wrought iron pipe was installed on High Bridge to carry water from the Croton Aqueduct, across the Harlem River, to New York.

In 1925, when plans for altering High Bridge were being discussed, the question arose whether the big water main was in a condition to be worth saving.

"Good for twenty years," reported the engineers after careful inspection. "Capable of being made to last another sixty years if a protective coating is applied to the inside."

Not a unique but a noteworthy example of the sterling qualities of genuine wrought iron pipe, in water works economy.

Genuine wrought iron is the material from which Byers Pipe has always been made. All the virtues for which Byers Pipe has been known to four generations are built into every length turned out by the Byers mills. The bright Spiral Stripe, which is the Byers mark, is an assurance of lasting service.

A. M. BYERS COMPANY
Established 1864
PITTSBURGH, PA.
Distributors in all Jobbing Centers

the *Spiral Stripe*

BYERS PIPE

GENUINE WROUGHT IRON

PIERCE-PERRY CO.

SUCCESSORS TO
GEORGE E. GILCHRIST CO.

JOBBERS OF
Heating and Plumbing Supplies
Steel, Wrought Iron and Brass Pipe
Water Works Materials

DISTRIBUTORS OF
Huffman Valves and Controlled Heat Equipment

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BOSTON PIPE & FITTINGS CO.

WHOLESALE JOBBERS

Byers Genuine Wrought Iron Pipe

National Pipe

COMPLETE LINE OF WATER WORKS SUPPLIES

273-279 CONGRESS ST., BOSTON

NOTICE:

Water-Works Contractors and Supply Houses!

Your card may be inserted here at a rate of $12 per year.

We Carry in BOSTON STOCK for Immediate Shipment

CAST IRON BELL AND SPIGOT WATER PIPE AND FITTINGS
FLANGED PIPE in full and short lengths
WROUGHT PIPE

FRED A. HOUDLETTE & SON
(Incorporated)

40 Central Street, Boston, Mass.

Quotations furnished promptly for shipment from Foundry

CEMENT LINED SERVICE PIPE

With Specially Adapted
Lead-lined Fittings

Also Curb Cocks which eliminate Iron to Brass Corrosion both inside and out.

CEMENT LINED PIPE CO.
LYNN, MASS.

SUMNER & DUNBAR

Manufacturers of and Dealers in
S. & G. Water Works Brass Goods
Byers' Wrought Iron Pipe
Service and Valve Boxes
Cast Iron Pipe and Fittings

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Telephone, Liberty 1350

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(Reg. U.S. Pat. Off.)

FIRE HYDRANTS

For Standard and High Pressure Service

Mathews Hydrants are simple in construction; hence durable, easily repaired, and maintained at low cost.

A real Frost Case makes Mathews Hydrants completely frost proof, and with an easily installed

Extension Section

makes the Mathews readily adaptable to new grades and conditions.

Water Works Gate Valves
Valve Boxes—Indicator Posts

CAST IRON PIPE FITTINGS

"Sand-Spun" centrifugally cast pipe
Cast in refractory molds
No chill—Standard bell and beaded spigot

R. D. WOOD & CO.

PHILADELPHIA, PA.

KENNEDY

VALVES~PIPE FITTINGS~FIRE HYDRANTS

WATER supply and fire protection equipment *must* be thoroughly dependable. Only apparatus of the highest possible grade, meeting the most rigid requirements, can be used.

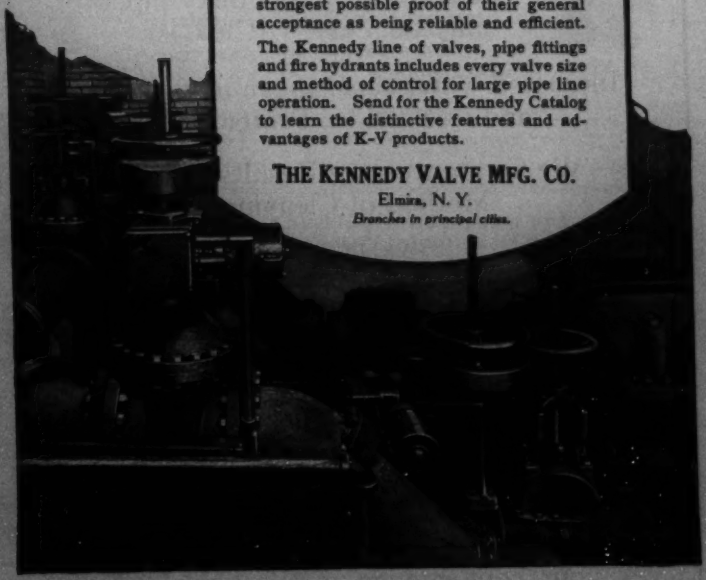
The wide use of Kennedy Valves and Fire Hydrants by villages, towns and cities throughout the country is therefore the strongest possible proof of their general acceptance as being reliable and efficient.

The Kennedy line of valves, pipe fittings and fire hydrants includes every valve size and method of control for large pipe line operation. Send for the Kennedy Catalog to learn the distinctive features and advantages of K-V products.

THE KENNEDY VALVE MFG. CO.

Elmira, N. Y.

Branches in principal cities.



Automatic Pressure Control Valves
Feed Water Filters
Portable Fire Hydrant Heads
Hydraulic Engines
Hydraulic Booster Pumps

ROSS VALVE MFG. CO., INC. TROY, N. Y.
ESTABLISHED 1879

Health and Protection First

Water delivered through dirty pipes may be a MENACE.

Incrusted water pipes mean inefficiency and loss of Fire Protection.

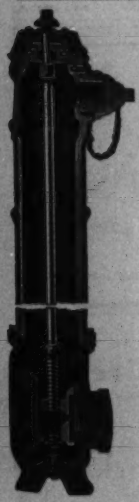
We Guarantee the Results of Our Method of Cleaning.

WRITE US.

National Water Main Cleaning Co.

50 Church Street New York City

A Classified Directory
of Advertisements is arranged for your convenience in the back of this Journal.



LUDLOW VALVE MFG. CO.

MANUFACTURERS OF

VALVES and FIRE HYDRANTS

This hydrant is anti-freezing, because when the drainage is good no water is left in it to freeze.

The drip is directly in the bottom of the hydrant and drains perfectly. It is protected by its valve, which never leaves its socket and cannot be clogged.

DOUBLE AND SINGLE GATE VALVES,

ALSO CHECK VALVES,

FOOT VALVES.

FIRE HYDRANTS.

YARD, WASH, AND FLUSH HYDRANTS.

☞ SEND FOR CIRCULARS ☜

OFFICE AND WORKS: FOOT OF ADAMS STREET, TROY, N. Y.

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62 GOLD ST.	HARRISON BLDG.	1112 TREMONT BLDG.	1ST NAT. BANK BLDG.	THE ROOKERY	R. A. LONG BLDG.

The Knuckle Joint Principle in Genuine "Rensselaer" Improved "Corey" Fire Hydrants—Means Ease of Operation

BECAUSE this principle gives great power to open or close the gate just when it is needed, and speed in movement when power is unnecessary.

The first turn of the spindle, when the gate is open, moves the valve nearly two inches, after which the speed decreases and power throughout the knuckle joint increases, until at the time of closing the valve moves scarcely one-sixteenth of an inch at each turn of the spindle, eliminating danger from water hammer when closing hydrant.

The gate is closed very tight with slight application of power to the wrench by the operator. The drip valve in the Corey Fire Hydrant prevents freezing. No accident or damage to property can occur with this hydrant by flooding the streets where runaway teams or other accident breaks off or otherwise injures the hydrant standpipe, if hydrant was properly closed at time of accident.

It will be readily seen that the hydrant gate is held in position when shut by the four arms forming braces between the back of the hydrant and the seat, consequently the hydrant barrel can be broken completely off above the ground, but the valves remain tight.

Write for copy of our Hydrant Book No. 9.

CHARLES L. BROWN,

New England Sales Manager, Lock Box 8, Northboro, Mass.

RENSSELAER VALVE COMPANY

TROY, N. Y.

THE CHAPMAN VALVE MANUFACTURING COMPANY

Manufacturers of

Gate Valves Drip Valves

Gate Fire Hydrants

Corporation Cocks Curb Cocks

Anderson Couplings

Lead Pipe Connections

Trade

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Indian Orchard, Mass.

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Type of Eddy Hydrant Popularly Used

More Than Sixty Years of Experience Goes Into All

EDDY

Hydrants and Valves

BUILT TO MEET EVERY EXACTING REQUIREMENT OF WATER-WORKS SERVICE

Practical water-works men instantly recognize the superior features of Eddy products. For instance, with the hydrant.

A damaged valve can be replaced in a few minutes — Water hammer is avoided by the valve closing against the pressure and the use of the cut-off under the valve, which gradually closes it —
Stems cannot be bent in closing the hydrant.
Nothing can lodge on the valve seat to prevent closing of the valve —
The Drip Rod can be removed without interfering with the hydrant in any manner —
The position of the stem of the hydrant indicates accurately the position of the valve —
Whatever can pass the valve opening will pass the standpipe —
No frost case necessary —
All of these points are explained thoroughly in our catalogue, which will pay you to study.

The Eddy Valve, Too

Has its distinctive and individual features. The double disk gates are free to adjust themselves in different positions every time the valve is closed, thereby keeping smooth and tight faces. And two bronze hooks on the ball loosely engage with the gates to prevent them from spreading at the top. The center bearing gates are forced to their seats with equal pressure at all points. There is a good deal more worth knowing about Eddy Valves. Further details will be sent for the asking.

Other Eddy Products

The Eddy line of water-works products also includes horisontal and vertical check valves, foot valves, hydraulically operated valves, indicator posts, etc.
All goods made by the Eddy Valve Company are manufactured exclusively at Waterford, New York.

WRITE FOR CATALOGUE

EDDY VALVE COMPANY

WATERFORD, N. Y., U. S. A.

EMAUS PIPE FOUNDRY,
DONALDSON IRON CO.,

MANUFACTURERS
... OF ...

CAST IRON PIPE

... AND ...

Special Castings for Water and Gas.
Also Flange Pipe, Street Castings,
Manhole Heads and Covers, etc.

EMAUS,
LEHIGH COUNTY, PA.

J. G. EBERLEIN, President and Treasurer.
J. M. GILBERT, Secretary.

CALDWELL "LEAKFINDR"

Listening for leak at hydrant

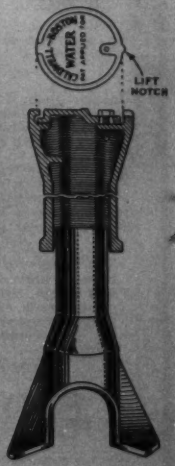
Locating leak in the ground

Pronounced by Leading Water Works Superintendents the Best Instrument for the purpose on the market.

Manufactured and Sold by

GEO. A. CALDWELL CO. - - MATTAPAN, MASS.

No. 1 CALDWELL CURB BOX



The FORD

STRAIGHT LINE YOKE

THE FORD Yoke is entirely *different in principle and operation* from any other meter holding device. Piping is held in *absolutely rigid alignment* and water-tight joints are made by *expansion from within* against rubber gaskets in fixed ports. In that principle lies *the undoubted superiority* of the Ford Yoke in all of its various types. The Straight Line Yoke is especially intended for setting water meters in straight service pipes whether in the cellar or in the meter box.

Expansion Connection

Every Ford Yoke includes the Ford Expansion Connection. This is an all bronze connection which is screwed onto the meter spud with a wrench. Meter is then dropped into the yoke between the gaskets and *turning the large hand wheel expands the connection outward* against the gaskets. Meter may be set or removed *without tools* even through a small, neat meter box opening.

The Angle Yoke

Here is another adaptation of the Ford Yoke and Expansion Connection principle. *Where the inlet is vertical and outlet horizontal*, this Angle Yoke should be used. It is made both plain or with stop valve as shown or with bleeder valve, and may be had with or without union. All of the inherent advantages of the Ford Yoke are obtained in any style of setting.

Ask for Bulletins

WATER METER SETTING & TESTING EQUIPMENT

THE FORD METER BOX CO.

WABASH, INDIANA

12 ADVANTAGES

1. *Meter may be installed or removed from the line in a few seconds at any time.*
2. *Costs but little, if any more than the meter couplings and pipe fittings it eliminates. Its cost is often saved in changing the meter once.*
3. *Pipe is maintained in perfect spacing and alignment whether meter is in or out. . Removal of meter does not move piping; no danger of causing leaks in old piping.*
4. *Meter is relieved of all piping strains.*
5. *Relieves troubles of wrong gasket space and different lengths of meters.*
6. *By making meters more readily available for removal and replacement it tends to make them better cared for and hence more accurate.*
7. *When meter is removed, it is difficult for customer to steal water because he cannot bridge across Yoke.*
8. *Yoke serves as a spacer, giving correct spacing and alignment. Meter may be set in a few seconds any time later. No necessity for pieces of pipe to be used as spacers and replaced with meter. Plumbing work on the outlet cannot destroy spacing or alignment, or put a strain on the meter.*
9. *The Expansion Connection is the only moving part of the Yoke. It is all bronze, and is removed with the meter. When the meter is out, there are no exposed threads to become corroded and fouled as is the case with ordinary couplings.*
10. *Yoke makes it easier to seal meters in line.*
11. *Permits the use of smaller and neater meter box lids.*
12. *For a few cents additional Yoke may be had with test or drain valve at outlet. Dead meters may be easily detected.*

Corrosion of Iron and Steel Water Pipes

Information on this subject is desired by the Committee recently appointed by the Association "to Collect and Correlate Information Concerning the Corrosion of Iron and Steel Pipes and Standpipes." Members are urged to send data relating to the characteristics of the metal, the soil, the water and the protective coatings and to the history and condition of such supply mains (not service pipes) to the undersigned for the use of the Committee.

CALEB MILLS SAVILLE, *Chairman.*
1026 Main Street, Hartford, Conn.

NICHOLAS ENGEL

JOHN FOX & CO.

CAST IRON

WATER & GAS PIPES

FLANGE PIPE

Special Castings, Fire Hydrants, Valves

General Foundry and Machine Work

WOOLWORTH BUILDING 233 BROADWAY
NEW YORK CITY

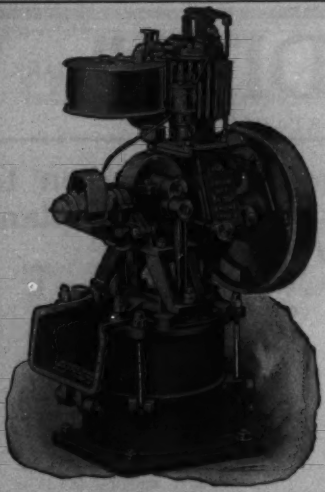
HIGHEST AWARD, GOLD MEDAL,
ST. LOUIS EXPOSITION, 1904

Over 100 Water Departments
use
LEAD-LINED IRON and TIN-LINED PIPES
for their service connections

MANUFACTURED BY

Lead Lined Iron Pipe Company

Wakefield — Mass.



STYLE "B" ENGINE – STYLE "A" NO. 3 PUMP
Furnished on Hand Skid or 4-wheel Truck

"On Top" Trench Pump Unit

EDSON

DIAPHRAGM
PUMP SPECIALTIES

Hand and Power Outfits

Try one of our Gasoline Two-Pump
EMERGENCY UNITS

Insist on GENUINE EDSON Pumps,
Suction Hose, Diaphragm
with Bead, etc.

New Catalogue on application

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Warren Foundry and Pipe Co.

(Formerly Warren Foundry and Machine Co.)

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201 DEVONSHIRE ST., BOSTON, MASS.
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CAST IRON PIPE

Bell and Spigot Flanged Pipe
Special Castings
Flexible Joint Pipe Cylinders

Water Gas Sewers Culverts

Works, Phillipsburg, N. J.

LARGE STOCK ENABLES US TO MAKE QUICK SHIPMENTS

TRADE **"LEADITE"** MARK

Registered U. S. Pat. Office

Only 51 days between the decision to build an emergency water works plant - - *and the finished job!*

- - the speed with which Leadite jointing can be accomplished contributed to this remarkable feat.

THE line illustrated here is 26,000 ft. of 12" pipe with LEADITE joints.

Owing to the emergency which arose in the city for which the installation was made no trenching could be attempted.

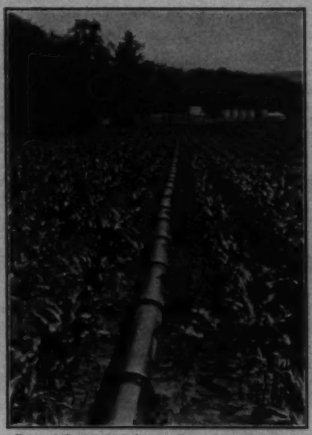
The line lies throughout its entire length on the surface of the ground without any bracing except at the bends.

After completion the line was tested to 260 lbs. pump pressure.

It is now under constant pump working pressure of 230 lbs. For over 30 years now, LEADITE has demonstrated by test and experience that it is absolutely permanent and secure.

Leadite is self-caulking and is so simple to use that the method of using it can be taught to even the most inexperienced help in a very short time.

Added to its speed and ease of use — Leadite saves at least 75% on account of saving in material and labor.



Part of the 26,000 ft. installation, crossing an open field. Chas. E. Waddell & Co., Engineers.

Specify and use LEADITE —it saves at least 75%

Be sure it is LEADITE and accept no imitations : :

The pioneer self-caulking material for c. i. pipe. Tested and used for over 30 years.

THE LEADITE COMPANY Land Title Bldg. PHILADELPHIA, PA.

NO CAULKING

Chadwick-Boston Lead Co.

800 Albany St., Boston

Agents for

The Celebrated "ULCO"

LEAD WOOL

(Every Atom Pure Lead)

For calking pipe joints under the most difficult conditions.

For overhead joints, or in wet places where the use of molten lead is not only impracticable but dangerous — **LEAD WOOL** may be used to advantage.

It makes an absolutely tight joint which will withstand the highest pressure, yet be sufficiently elastic to allow considerable sagging or settling of the pipe without danger of a leak.

As compared with the poured joint, in this respect, the superiority of **LEAD WOOL** is apparent.

Manufacturers of

Lead Pipe, Tin Lined Lead Pipe, Pure Block-Tin Pipe, Solder, White Lead and Red Lead.

HIGHEST QUALITY SOFT BRANDS

PIG LEAD

Joints that are Stress and Vibration Proof

Unusual situations often met in laying bell and spigot pipe — lines over bridges or railroad crossings — lines to be lowered to grade — are successfully met when the joints are made with Hydro-Tite.

Easy to Prepare

The 8-in. line pictured above is slung across a pond with cables — a span of more than fifty feet.

Every test shows that Hydro-Tite joints will remain perfectly tight when subjected to severe vibration or deflection. If you have never used Hydro-Tite let us tell you more about it. Requires no caulking — 19 years of proven dependability — a real 75% saving and an ever-increasing list of satisfied users.

Easy to Pour

Write for literature and further information.

HYDRAULIC DEVELOPMENT CO.

50 Church St., New York, N. Y.

BOSTON OFFICE & WORKS
296 Boylston St.

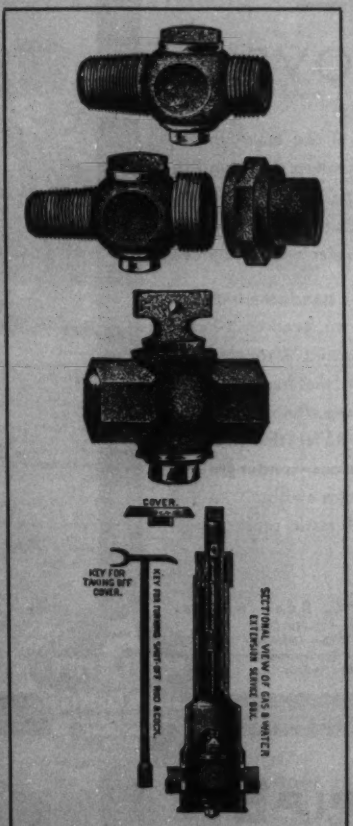
CHICAGO OFFICE
7 So. Dearborn St.

HYDRO-TITE

REG. U. S. PAT. OFF.

The Modern Joint Material

"The Goods That Please"



Corporation and Curb Cocks

We have them to meet every requirement

ORIGINAL

"Hays-Erie"

Extension Service Boxes of proven advantages

Let us send at our risk, on 30 days' trial, a

Payne's Patent Tapping Machine

which is recognized to be the most easily operated owing to the few working parts.

Hays Mfg. Co., ERIE PENNSYLVANIA

Established 1869

UNDER GROUND— OR ABOVE

When pipe resists all five of the major forces that tend to lower pipe endurance, it's bound to be good. That's what Reading 5-Point Pipe does —and that's why Reading 5-Point Pipe effects such remarkable economies in water works service!

Behind Reading 5-Point Pipe stands the indisputable record of the years—for Reading 5-Point Pipe is made of Genuine Puddled Wrought Iron, the same time-tested material that has proved its worth through long ages of use. An overwhelming mass of evidence attests the fact that Reading 5-Point Pipe lasts for generations—under ground or above! The Reading 5-Point emblem and the Reading cut-in spiral knurl mark protect you from substitutes.

READING IRON COMPANY, Reading, Pa.

Atlanta · Baltimore · Cleveland · New York · Philadelphia
Boston · Cincinnati · St. Louis · Chicago · New Orleans
Buffalo · Houston · Tulsa · Seattle · San Francisco
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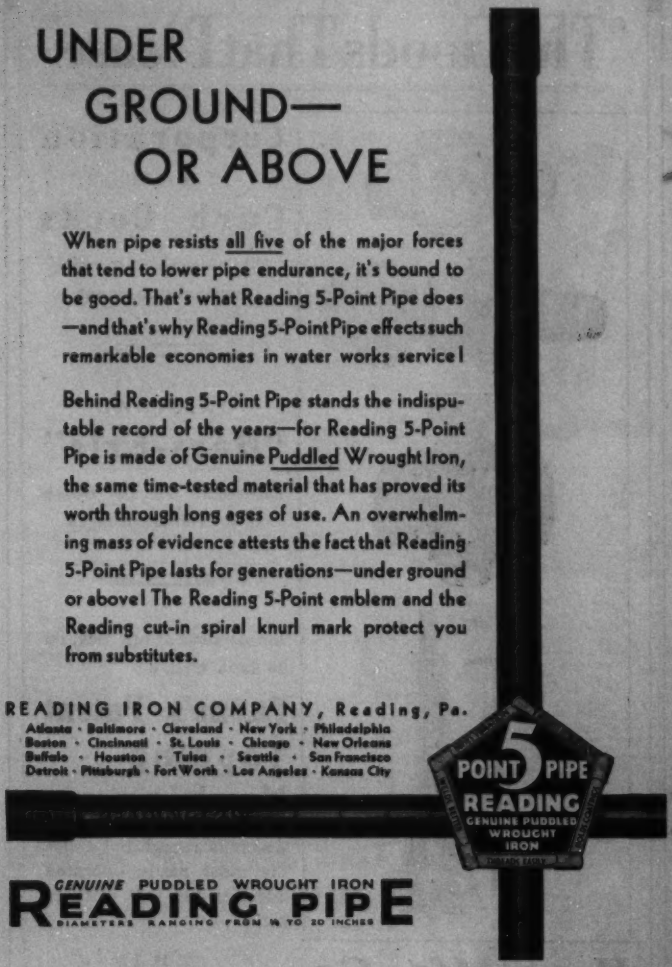


GENUINE PUDDLED WROUGHT IRON
READING PIPE
DIAMETERS RANGING FROM 4 TO 20 INCHES

GOLDEN-ANDERSON

Patent Automatic Cushioned Controlling Altitude Valves

We have enlightened many a water user in municipalities, in industrial plants and on railroads, as to the adaptability of these Golden-Anderson Valves, and as to their function of SAVING, and of eliminating troubles.

For automatically maintaining Uniform Stage of Water in Tank, Reservoir or Standpipes. Doing away with the annoyance of Freezing and Float Fixtures inside or outside of Tanks.

"Three Ways of Closing These Valves":
1st —Automatically, by water.
2nd—By Electricity, if desired.
3rd —By Hand

May also be arranged to automatically close when a break occurs in the mains. When necessary they may be so connected as to "work both ways" on a single line of pipe. No valves or fixtures inside or outside.

Sizes to 24 in.

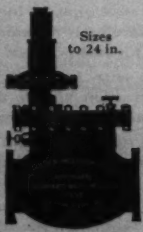
Remember!

"HOSTS OF REFERENCES"
Valves cushioned at all times by air and water
No water hammer or bursting mains

"Made with stop starter attachment for centrifugal pumps"

GOLDEN-ANDERSON
Pat. Automatic Cushioned Water Pressure Regulating Valves

1. Maintain a constant reduced pressure regardless of fluctuations on high pressure side.
2. Perfectly Cushioned by water and air. No metal-to-metal seats.
3. The best valve made for maintaining a constant low pressure where consumption is continuous.
4. Operates quickly or slowly as required—No attention necessary.
5. Positively no hammering or sticking.



Sizes to 24 in.

GOLDEN-ANDERSON
Patent Automatic Cushioned Controlling Float Valves

They carry a constant water level in feed water heaters by perfectly controlling the flow of makeup water.

Operated by protected enclosed copper float.

Cushioned by both air and water.

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5. Cushioned by Water and Air.

Angle or Globe

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Sizes to 24 in. Angle or Globe

Automatically prevents reverse flow of pressure.

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Arranged so that quick or slow operating may be obtained.

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Testing and loading department at Biggs Plant.

Background—Pipe being vertically hot dipped.

Foreground — Special section with angle anchorage and reducer with cast steel flange for line valve connection.

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Steel Pipe

is worthy of your confidence

The superior structural strength of steel pipe, its long life and ability to give continuous and uninterrupted service is responsible for the preference that is continually being shown for this product.

Biggs Electric Welded Steel Pipe with its 100% joint efficiency, smooth interior and standardized fabrication affords the Water Works Engineer an ideal conduit for major and important projects.

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Although it costs a trifle more in the beginning, pipe or tubing of Copper or Brass is *always* an economy in the long run. Its use eliminates rust and rust-clogging that inevitably result from corrosion. Properly installed, Copper or Brass gives lasting satisfactory service.

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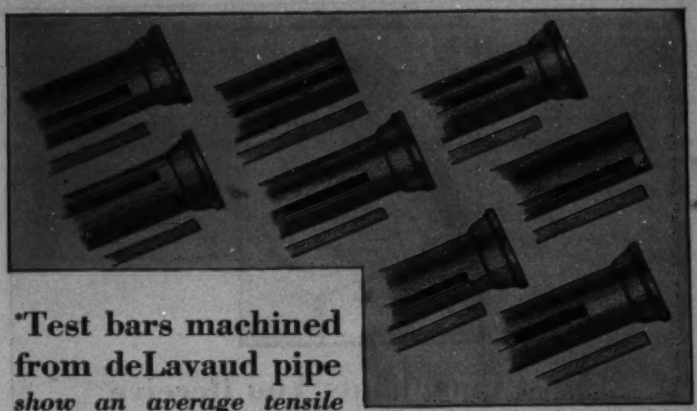

"Test bars machined from deLavaud pipe *show an average tensile strength of more than* 17 tons per square inch

* "The tensile strength of bars machined from deLavaud pipe averaged 35,658 lbs. per square inch. This figure is approximately *100%* higher than that obtained from good pit cast pipe." (Report of actual test conducted by an outside Testing Laboratory. Details on request.)

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In hydraulic bursting tests, deLavaud Pipe also showed a greater strength than pit cast. In these tests, deLavaud Pipe withstood an average internal pressure of more than 3,000 lbs. per square inch or 25% higher reading on the guage than pit cast pipe.

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MAKING pH determinations with the improved LaMotte Roulette Comparator is accurate and rapid; in fact, it is no more difficult than measuring temperatures with a thermometer. Notice its simplicity — the arrangement of the color standards in the revolving drum — the special lighting arrangement, so that the observer always makes his readings under standard daylight conditions.

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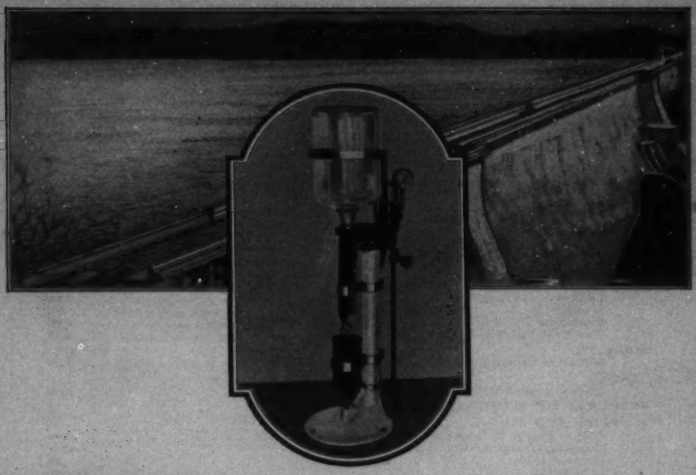

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